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STAFF REPORT

Future Adequacy of Sewage Collection System For Area East of Stevens Creek

Submitted by
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Director of Public Works
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TABLE OF CONTENTS

CHAPTER 1

INTRODUCTION AND SCOPE OF STUDY

Introduction

In September 1973 Brown and Caldwell, Consulting Engineers, completed a study and report on the City's sewage collection system. Part of the study included the development of updated flow factors to use in estimating future sewage volumes to be expected from various land use zones throughout the City. These flow factors were arrived at largely by analyzing water use records for the various land use zones. Based on these flow factors, the Brown and Caldwell report projected that in the industrial area east of Stevens Creek, the existing sewers will be deficient in capacity to handle projected peak sewage flows at full development.

Since about 400 acres of industrial land are still undeveloped tributary to the East Trunk sewer which serves this area, major sewerage improvements were recommended by Brown and Caldwell to meet the projected needs. This work includes over $600,000 worth of sewers to handle greater flow than for which the system was originally designed. The additional flows would also require purchase of additional capacity rights in the Regional Water Quality Control Plant.

Currently, there is considerable interest to develop acreage east of Route 237 where no sewers have been constructed. Sewers to serve this area would connect to the East Trunk sewer system.

This study is basically to determine the affect of this and other future development and to recommend a basis for future sewer design.

This report has been prepared by N. H. Lougee, Water Division Engineer, L. F. Janda, Design Engineer and R. J. Irwin, Jr. Civil Engineer.

Scope of Study

On February 25, 1974, the City Council authorized the Director of Public Works to undertake an engineering study to assess the adequacy of the East Trunk sanitary sewer system with respect to existing and full development conditions. The study area is shown on Figure 1-1.

Scope of study includes the following:

1. Establish volumes of wastewater flow presently being generated by industrial activity and precise location of the points of discharge.

2. Determine water use and sewage discharge patterns to establish times of peak discharge.

3. Determine reduction in sewage flow necessary to assure continued future use of existing sewers without the need for constructing new mains.

4. Determine if revenues generated by sewer rental charges cover the cost of conveying and treating the industrial wastewaters.

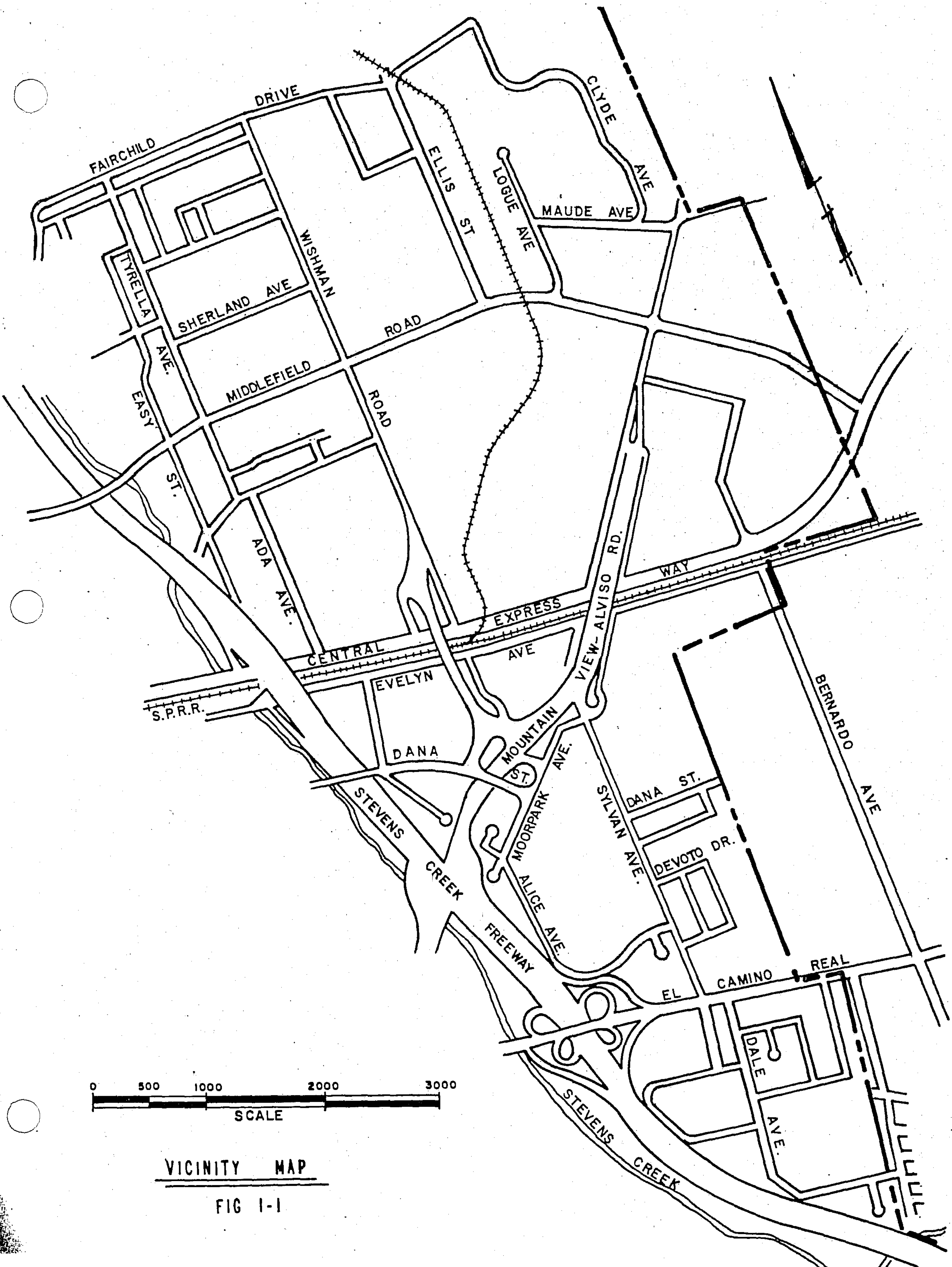
5. Prepare preliminary rate adjustments if needed.

6. Visit industries and discuss sewerage needs and costs and ask

for projections of future sewage discharges. Discuss possible reductions in peak and total flows and timing of large discharges as appropriate.

7. Based on the above, determine if continued operation of the existing system is possible and for how long. Establish what possible measure should be pursued to accomplish a least cost solution consistent with providing adequate service. Such measures should include revising rates to cover all associated costs and might include major water conservation and reuse programs within industries, rescheduling of activities to stagger peak discharges, construction of relief sewers if sewage flows cannot be reduced enough, or establishing by ordinance maximum flows or water use which will be allowed.

8. Submit a report making recommendations to the City Council for approval to implement the recommended plan.



VICINITY MAP

FIG 1-1

CHAPTER 2

EXISTING WASTEWATER FLOWS

Measurement of Existing Wastewater Flow

This portion of the study was undertaken to establish the physical characteristics of the existing system, to establish times of peak flows, determine existing flows and to verify Brown and Caldwell's unit flow factors for industrial developments. Since many manholes in the area are deep and irregular and surcharging was expected, the most practical method of measuring wastewater flows was determined to be by measuring depths of flow and converting these depths to a quantity of flow per unit time.

The depth measurements were accomplished with two city depth recording gauges and two gauges rented from Manning Environmental Corporation. These gauges were placed in 11 manholes for periods ranging from two to four days. Locations were chosen which allowed correlation of flows to individual industry discharges and which had a minimum of turbulence. The majority of measurements were between mid-March and late April during a period of low rainfall.

To establish a relationship between depth and quantity of flow, velocity measurements were also taken at this time. Small fluorescent balloons were floated and timed from the manhole upstream of the one being gauged to the one downstream, where possible. Water was added to the balloons so they would float beneath the surface but off the bottom to permit a more accurate determination of average velocity than would a slower moving surface float. Depth of flow measurements were taken in conjunction with the velocity measurements.

The following data was obtained by the field measurements:

1. Depth of flow versus time as shown in Figure 2-1.
2. Velocity of flow versus depth of flow.

Using the velocity data, theoretical slopes for each pipe were calculated using Manning's formula. The formula used and an example calculation is included in the Appendix. An "n" factor of 0.011 was used in these equations since it was found that slopes derived using this factor were characteristic of existing conditions. This value is not unrealistic to expect if wastewater does not have appreciable amounts of grit, debris or other solids. The sewage discharged by the industries in the study area is typically clean water. The sewers were originally designed using a more conservative n=0.013.

The calculated slopes were then used in the Manning equation, with values for area and hydraulic radius calculated from the measured flow depths, to arrive at flow quantities in million gallons per day (MGD). The depth of flow versus time curves were then converted to give flows plotted against time for each measuring station. These flow graphs are also given in the Appendix. Estimated flows for pipes flowing full under gravity were computed in the same way by setting the flow depth equal to pipe radius and multiplying the corresponding flow by 2. All measured quantities were assumed to be dry weather flows since there was little rain during the periods manholes were monitored.

Verification of Measurements

To verify the flow measurements, a comparison of the measured flows was made with calculated existing flows based on metered water use for industrial areas and on flow factors developed by Brown and Caldwell for residential and commercial areas. Water use by industries was obtained from the Finance Department's utility ledgers for the period from March to April 12, 1974. The total used by each industry between meter readin was divided by 27 days to obtain average daily water usage (weekend wate use assumed at 50% of weekday use).

To estimate peak dry weather wastewater flows for industrial activities a peaking factor of 1.5 was used. This factor is contrary to the peaking factor of 3.0 used by Brown and Caldwell for industrial discharges in their report but was selected for the following reasons:

1. The type of industrial activity in the study area is primarily electronic equipment production and typically does not have a wide fluctuation in amount of wastewater discharged.

2. Fairchild Semiconductor's peak wastewater discharge is close to 50% greater than their average wastewater discharge. Fairchild is by far the largest discharger and therefore, has the most influence over the peaks that occur.

3. The graphs plotted from field measurements indicated that 1.5 was a realistic peaking factor.

Peaking factors for other land uses were assumed to be as recommended by Brown and Caldwell.

Based on this data, existing sewage flows are shown in Table 2-1 for each sewer sub area. These sewer sub areas and location of flow measurements are shown on Figure 2-2. Figure 2-2 also shows the measured peak dry weather flows at locations where flows were measured as well as calculated peak dry weather flows at other locations. Sewer capacities are also shown on Figure 2-2 based on pipes flowing full with no surcharge. The measured and calculated peak flows compare well for sewers that serve sub areas that are mainly industrial or residential but differ for sewers that serve both types within a sub area. This is due to the difference in characteristics of residential and industrial sewage flows.

Figure 2-3 shows typical industrial and residential flows for a 24 hour period based on measured flows in the Fairchild Drive and Tyrella Avenue sewers. It can be seen that the peak flow in the Fairchild Drive line (industrial) occurs at a different time than the peak flow in the Tyrella Avenue line (residential).

Analysis of Existing Wastewater Flows

The Brown and Caldwell report concluded that the average wastewater unit design flow factor for limited industrial zoning should be 5,000 gallons per gross acre per day with a peaking factor of 3. This unit design factor is based on water usage and represents an average value derived by analyzing several industrial areas within the City.

Calculated peak industrial flows for the study area compare well with this factor. Calculated average flows, however, are considerably higher than the Brown and Caldwell design factor. Based on water usage for the period March 12 to April 12, 1974 by industries representing 309 net acres within the study area, it was determined that an average wastewater volume unit design factor of 11,000 gallons per net acre per day would be appropriate for existing conditions. Applying a peaking factor of 1.5 as previously discussed in this chapter produces a peak dry weather flow factor of 16,500 gallons per net acre per day. Allowing for a reduction of 20% for street areas, Brown and Caldwell's peak dry weather flow factor based on net area would be 18,500 gallons per acre per day.

From Figure 2-2B it can be seen that on Fairchild Drive between Ellis Street and Whisman Road, the calculated and measured existing peak flows are greater than the sewer capacities, as follows:

Location	Size	Sewer Capacity (M.G.D.)	Existing Peak Flow (M.G.D.)
Fairchild (Ellis to National)	18"	1.82	2.32
Fairchild (National to Whisman)	18"	2.20	3.91

FIGURE 2-1

TYPICAL GRAPH INDICATING DEPTH OF FLOW AGAINST TIME

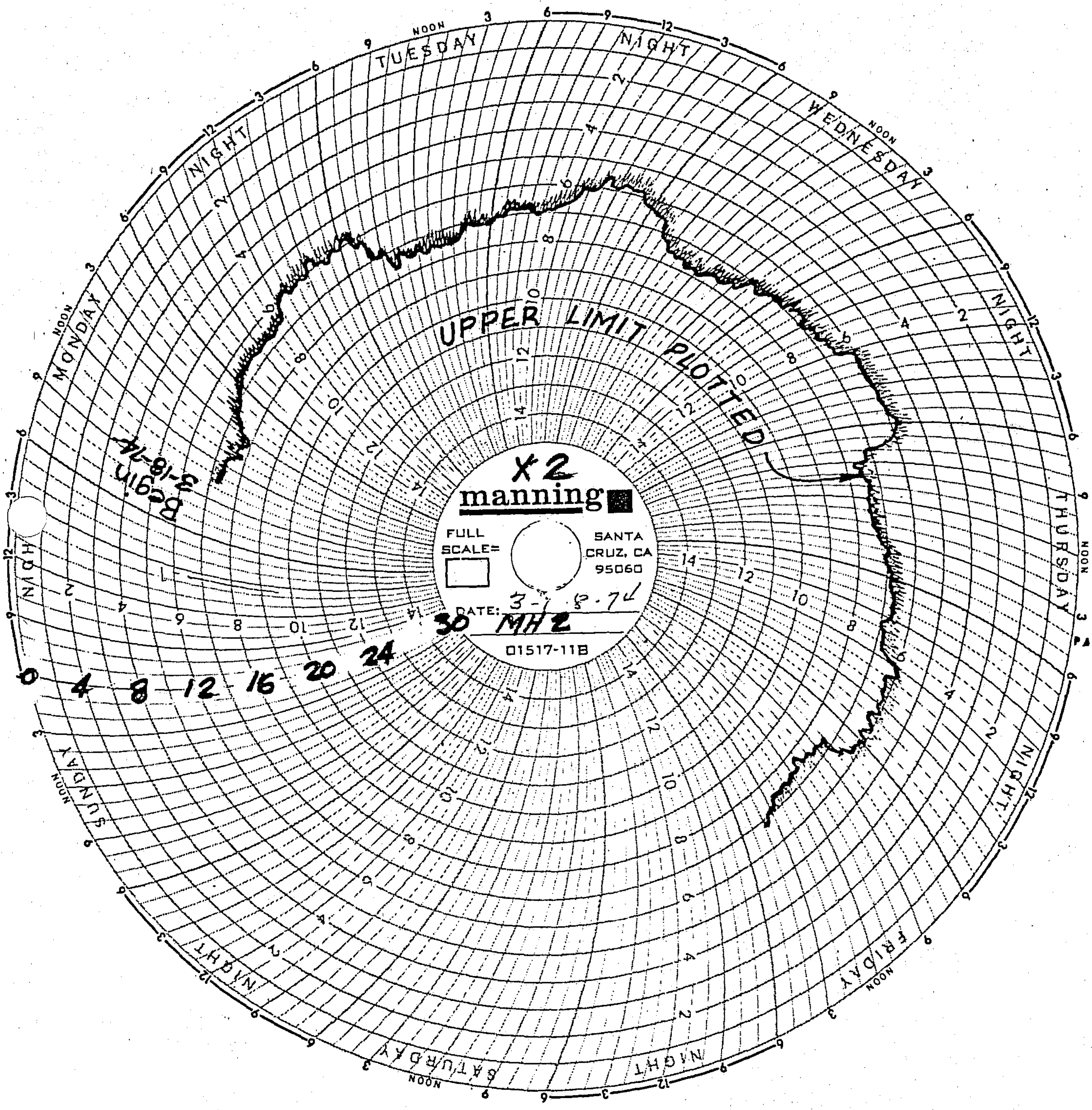


TABLE 2-1 — EXISTING WASTEWATER FLOW

SUBAREA	LAND USE		NO. UNITS	AREA ACRES	ADWF M.G.D.	PDWF M.G.D.	PSF M.G.D.
Americana	Residential	SF					
		MF	695	29.44	.1100	.2800	.0290
	Commercial			12.56	.0070	.0210	.0100
Dale	Residential	SF	3	23.67	.0008	.0024	.0240
		MF	454		.0750	.1870	
	Commercial			9.40	.1034	.3102	.0090
Sylvan	Residential	SF	5	61.47	.0014	.0042	.0600
		MF	889		.1467	.3373	
	Commercial			4.81	.0050	.0150	.0050
Moorpark	Residential	SF	43	27.08	.0120	.0360	.0300
		MF	337		.0560	.1440	
	Commercial			0.80	.0009	.0027	.0008
Evelyn	Industrial			10.29	.0900	.1350	.0103
Tyrella	Residential	SF	183	167.00	.0510	.1430	.1670
		MF	1109		.1830	.4210	
	Industrial			37.77	.2433	.4000	.0378
	Schools		1	14.87	.0003	.0009	.0150
Whisman	Residential	SF	46	63.60	.0129	.0386	.0640
		MF	519		.0856	.2141	
	Industrial			41.58	.5360	.8090	.0042
	Commercial			1.03	.0011	.0033	.0010
	Schools		1	8.50	.0003	.0009	.0085
Fairchild	Industrial			10.17	.5900	.8800	.0102
National	Industrial			17.55	.4766	.7149	.0175
Clyde	Industrial			12.17	.0244	.0367	.0122
Ellis	Industrial			79.92	1.2820	1.9230	.0799
Logue	Industrial			52.63	.2380	.3540	.0526
Sylvania	Industrial			46.57	.1640	.3020	.0466
			TOTALS	732.88	4.50	7.71	.69

M.G.D. Million Gallons Per Day
ADWF Average Dry Weather Flow
PDWF Peak Dry Weather Flow
PSF Peak Storm Flow @ 1,000 G.A.D.
SF Single Family
MF Multiple Family

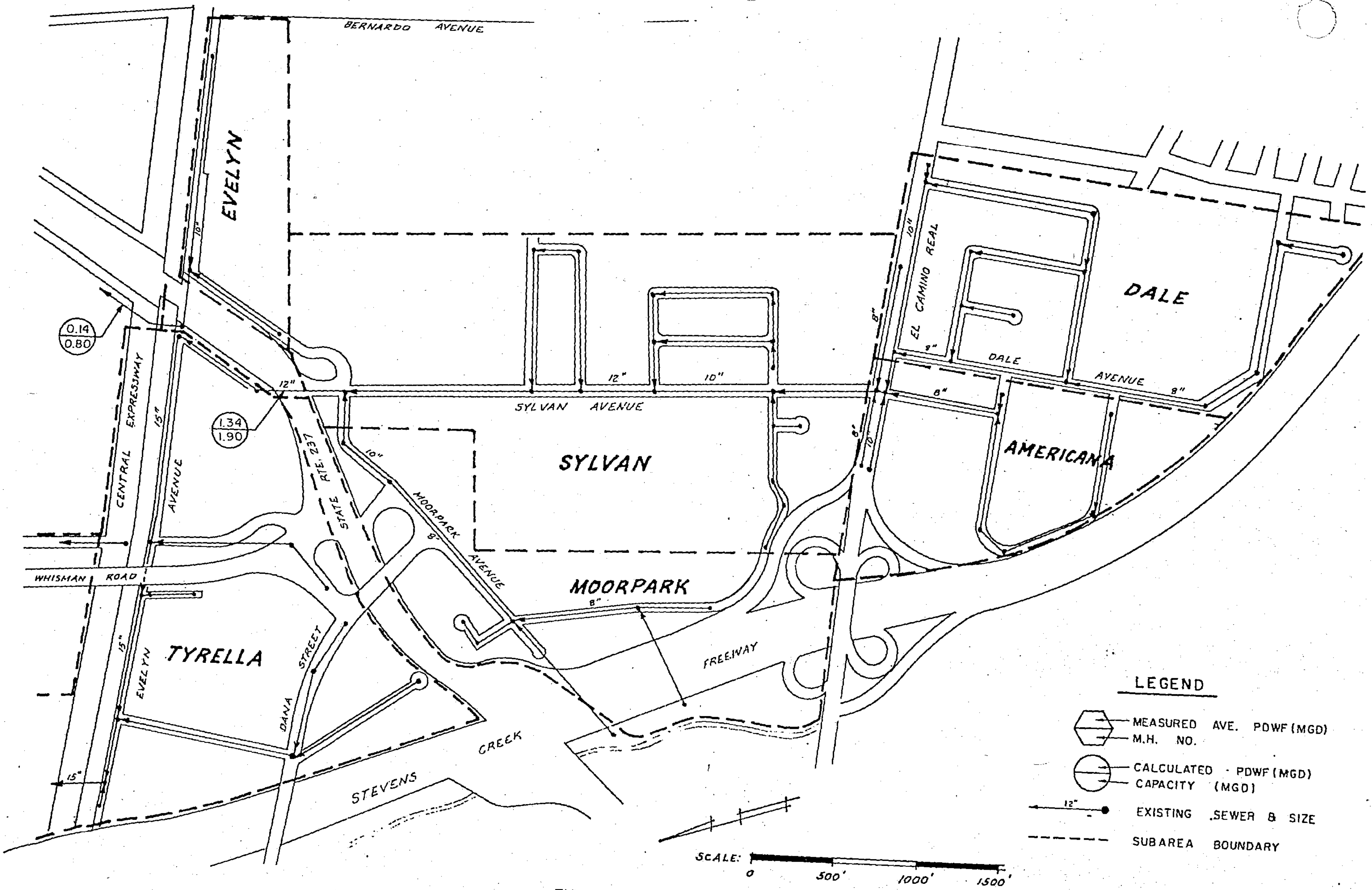


FIG. 2-2A

EXISTING MEASURED AND CALCULATED WASTEWATER FLOW

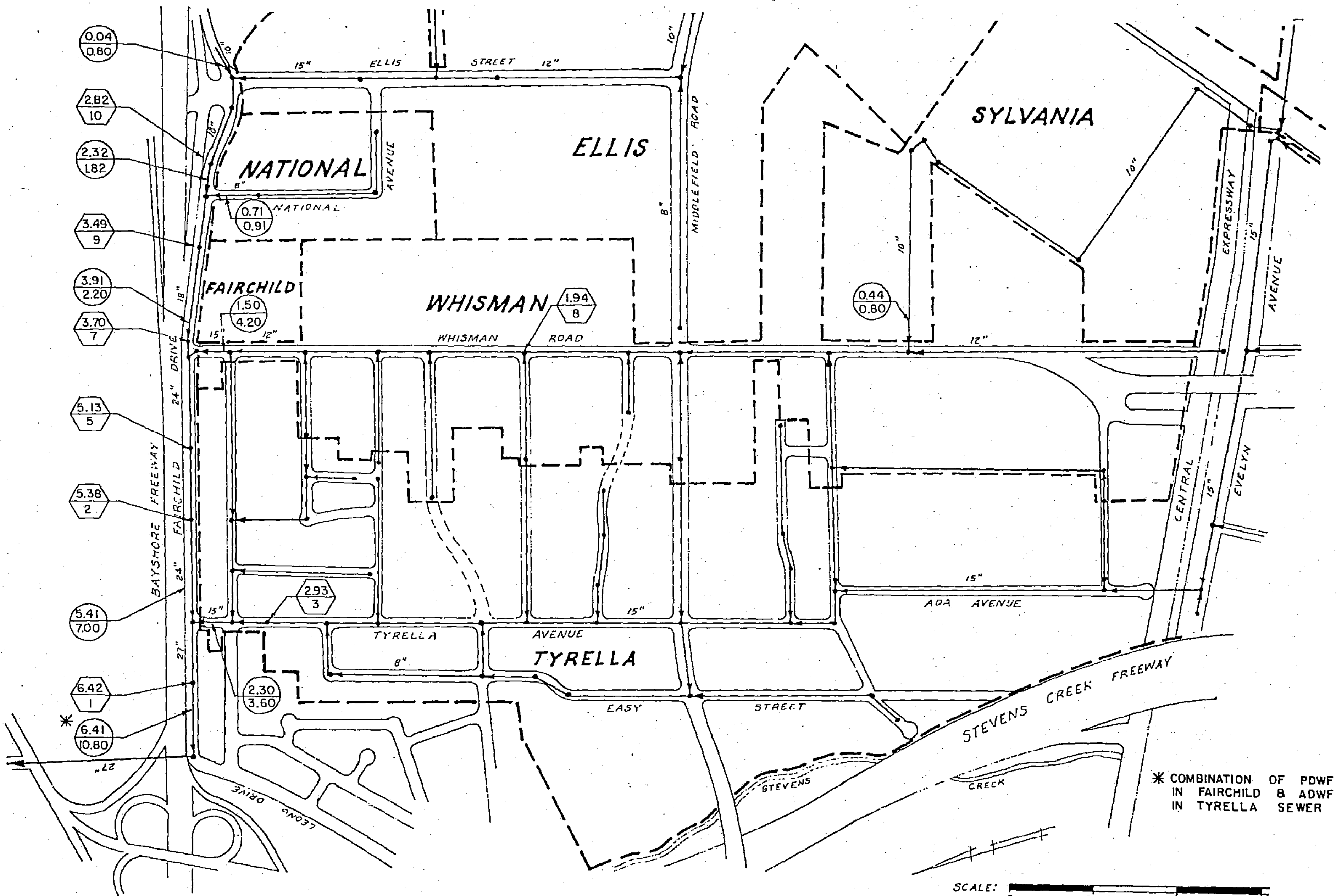


FIG. 2-2B
EXISTING MEASURED AND CALCULATED WASTEWATER FLOW

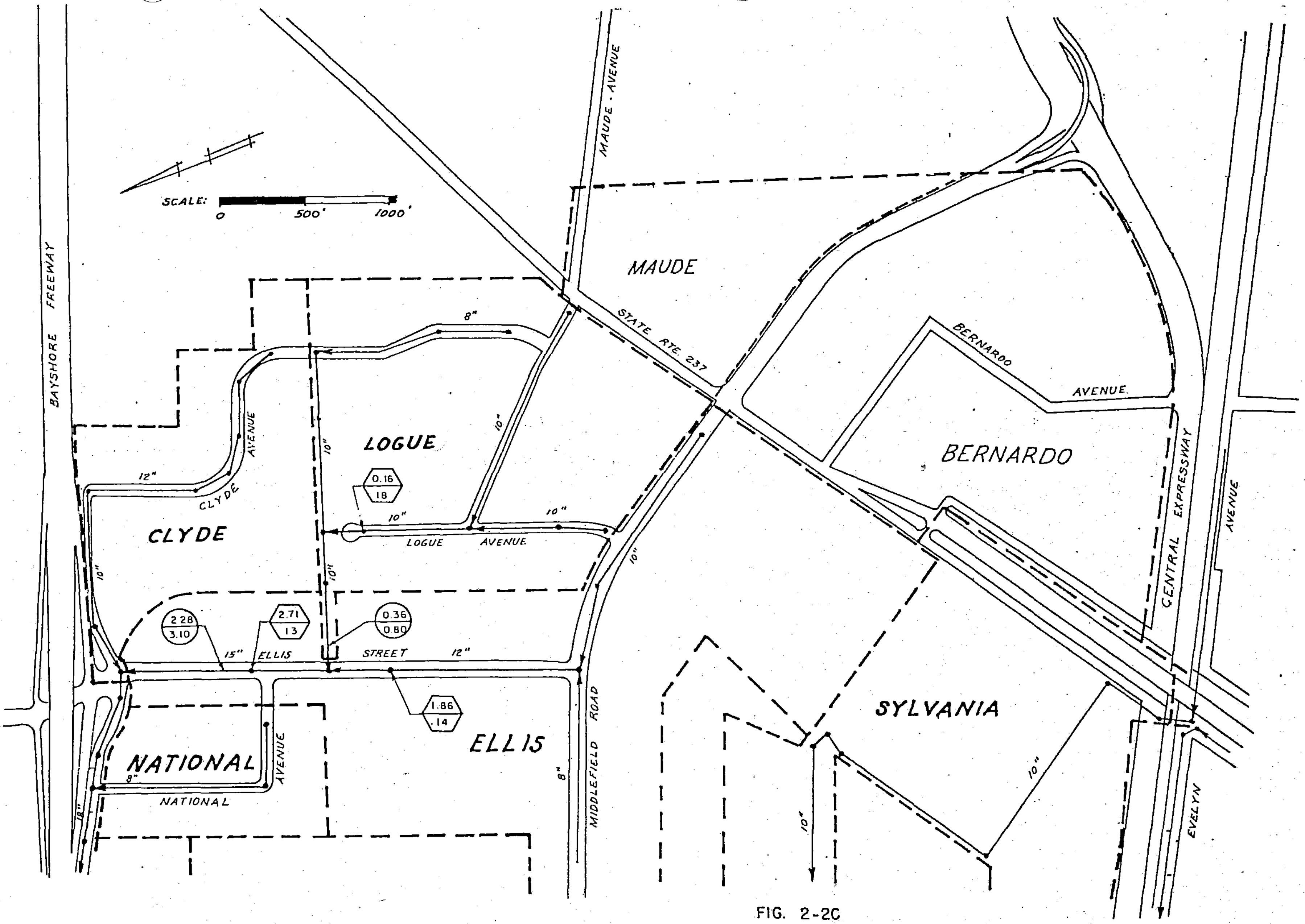


FIG. 2-2C
EXISTING MEASURED AND CALCULATED WASTEWATER FLOW

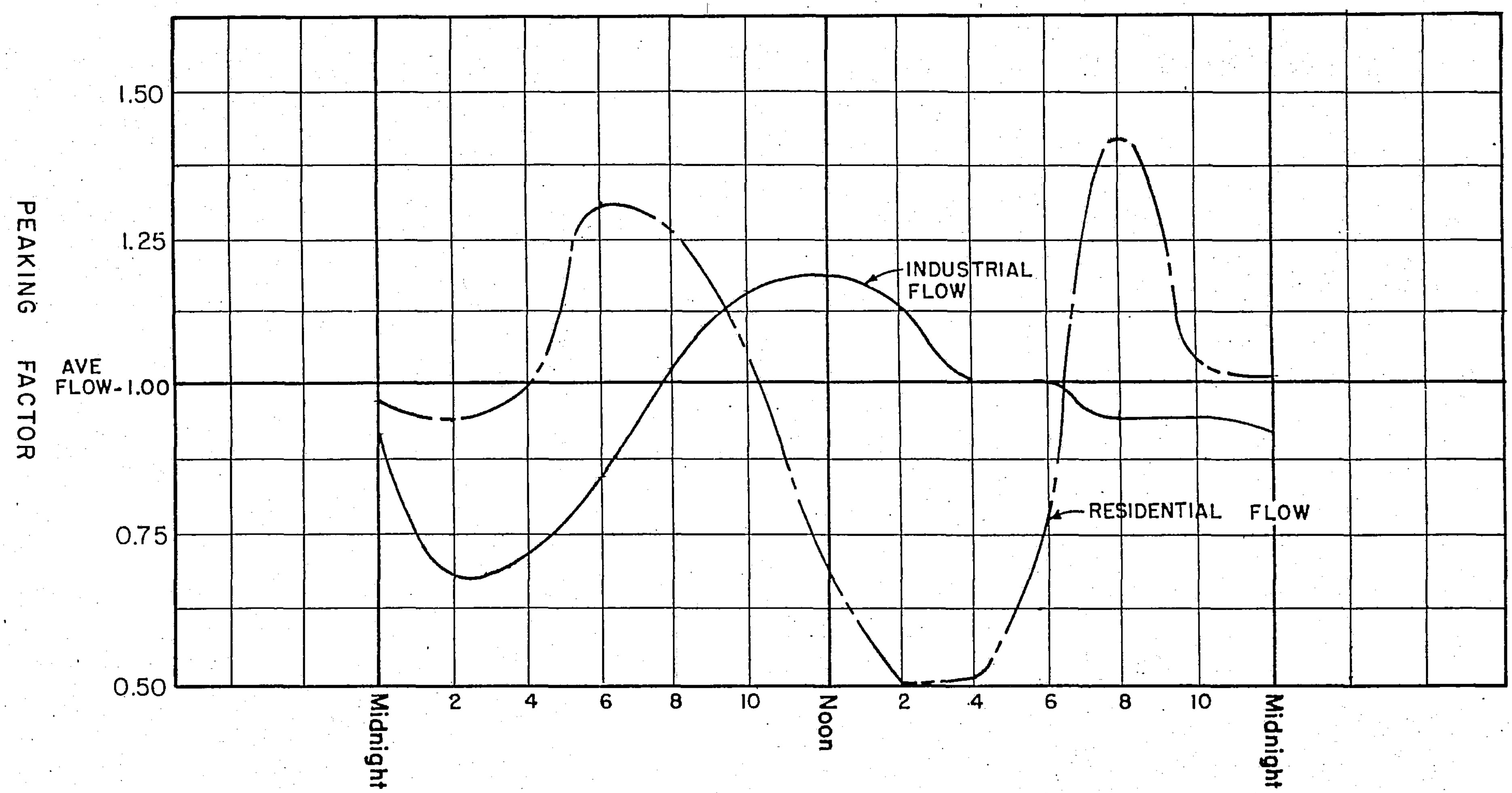


FIG. 2-3
TYPICAL DAILY FLOW VARIATION IN STUDY AREA

CHAPTER 3

ADEQUACY OF SEWER SYSTEM

Expected Future Flows

Based on an average wastewater flow of 5,000 gallons per acre per day (g.p.a.d.) and a peak dry weather flow rate of 15,000 g.p.a.d. and full development, flows in the sewer system tributary to the East Trunk sewer will be as shown on Figure 6-1 of the Brown and Caldwell report, a copy of which is included in the Appendix. These flows exceed the design capacity of many sewers in the area and are the basis for Brown and Caldwells recommended plan calling for over $600,000 worth of new sewer construction.

Based on our measurements, these flow rates are reasonable estimates assuming that no restrictions are placed on industrial waste discharges and that future industrial development will be of a type similar to past development.

Capacity of the System

The East Trunk sewer system which serves the land area east of Stevens Creek was constructed about ten years ago. This system was designed to handle peak industrial and commercial discharges of 6,000 gallons per acre per day, residential discharges of 70 gallons per capita per day and an allowance for infiltration of 1,000 gallons per acre per day.

The system design capacity, based on sewers flowing just full and on the above unit flow averages about 9,500 gallons per net acre per day.

Several industries now exceed these flow rates at peak flow and their discharges have already caused some local sewers to surcharge slightly to handle these greater than design flows. These excess flows have been offset somewhat, however, due to recent zoning changes which have reduced potential future development north of the Bayshore Freeway and which consequently have reduced the total flow expected in the East Trunk sewer. Moreover, existing high flow industries are located far enough downstream in the service area that the sewers can handle these excess flows. Such discharges if they occurred in upstream areas, would overload the sewers and in some areas could cause overflowing in the streets. Fortunately, most of the industries located in the upstream areas are currently discharging less than design flows.

Adequacy of Sewer System

As shown by Brown and Caldwell, the existing system will be overloaded if future industrial development equals existing development in terms of water use. Should industrial expansion continue as in the past decade, the major modifications to the sewerage system recommended by Brown & Caldwell will indeed be needed.

Existing industries, however, expanded during a period when resources appeared to be unlimited. Today, a different industrial climate exists. Industries now know that costs for water and wastewater treatment are increasing sharply. Furthermore, new Environmental Protection Agency (EPA) guidelines now require industries to pay for their share of federally funded sewerage projects. Industrial processes are also currently being scrutinized by the EPA to eliminate wasteful uses of water, power, and raw materials and to eliminate pollution.

It is now reasonable to expect that future industrial development will proceed in a manner that conserves resources wherever feasible. With this in mind, construction of new sewers on the basis of current wastewater discharge patterns may be uneconomical and might encourage wasteful water use practices.

The system as installed today is capable of handling existing flows with small amounts of local surcharging during short peak flow periods. The existing sewers can also basically handle flows from further development of the area if future peak flows from industrial development remain close to the original 6,000 g.p.a.d. design values.

Table 3-1 gives the projected wastewater flows from all currently undeveloped areas (about 400 acres) served by the East Trunk sewer system. Based on an industrial peak dry weather flow of 6,000 g.p.a.d. from these areas the flows shown may be combined with those in Table 2-1 to obtain total flows for each sub area and for the total area.

The peak flows including storm water infiltration which would result from the combination of existing and future development as shown on Tables 2-1 and 3-1 are given on Fig. 3-1. These peak flows are peak wet weather flows and consist of peak dry weather flows plus assumed infiltration. This figure also gives existing and future sewer capacities for comparison.

Figure 3-1 shows that most sewers have sufficient capacity to handle these peak flows. The notable exception is the East Trunk sewer in Fairchild Drive from Ellis Street to Whisman Road and to a lesser extent on Ellis Street. This trunk sewer, however, can tolerate a large surcharge as shown on Figure 3-2.

Figure 3-2 shows the hydraulic grade line * of the East Trunk sewer for the above peak flows and shows that surcharging based on these future flows would reach a maximum of about 3.5 feet. At this condition, the only tributary sewer that would discharge its sewage flow into the trunk sewer below the hydraulic grade line would be the National Avenue sewer. The resulting surcharge in the National Avenue sewer would not be a problem, however, since the sewer is well below all building connections and was constructed at a steeper than normal slope.

Figure 3-2 also illustrates the hydraulic grade line for peak flows predicted by Brown and Caldwell. The surcharging that would result from these peak flows clearly would affect tributary sewers adversely and would cause overflows from the East Trunk sewer upstream of Whisman Road.

* Footnote - The hydraulic grade line represents the surcharge level or height to which water will rise in sewer manholes at given flow conditions.

TABLE 3-1 — PROJECTED WASTEWATER FLOW FROM UNDEVELOPED AREA

SUBAREA	LAND USE		NO. UNITS	AREA ACRES	ADWF M.G.D.	PDWF M.G.D.	PSF M.G.D.
Americana	Residential Commercial	SF MF	175	2.50	.0288	.0866	.0025
Dale	Residential Commercial	SF MF	116	9.56 0.35	.0191 .0003	.0574 .0006	.0095 .0003
Sylvan	Residential Commercial	SF MF	114 312	19.06 28.37 14.60	.0319 .0515 .0035	.0958 .1360 .0056	.0191 .0284 .0146
Moorpark	Residential	SF MF	57 -63	8.08	.0160 -.0104	.0479 -.0312	.0081 -
Tyrella	Residential Industrial	SF MF	370	33.59 28.22	.0610 .1129	.1587 .1693	.0336 .0282
Evelyn	Commercial Industrial			1.93 5.62	.0021 .0225	.0064 .0337	.0020 .0056
Sylvania	Industrial			24.43	.0977	.1466	.0244
Whisman	Residential Commercial Industrial	SF MF	102	9.28 4.20 41.77	.0168 .0042 .1671	.0505 .0126 .2506	.0093 .0042 .0418
Maude	Industrial			31.75	.1270	.1905	.0317
Logue	Industrial			4.53	.0181	.0272	.0045
Bernardo	Industrial			70.08	.2803	.4205	.0701
Ellis	Industrial			41.37	.1655	.2482	.0414
Clyde	Industrial			16.76	.0671	.1006	.0168
National	Industrial			3.80	.0152	.0228	.0038
			TOTALS	399.85	1.2982	2.2369	.3999

M.G.D. Million Gallons Per Day
ADWF Average Dry Weather Flow
PDWF Peak Dry Weather Flow
PSF Peak Storm Flow @ 1,000 G.A.D.
SF Single Family
MF Multiple Family

IND. = 4000 GAD (AVE)
P.F. = 1.5

NOTE: Industrial Peak Flow Factor = 6,000 Gallons Per Acre Per Day

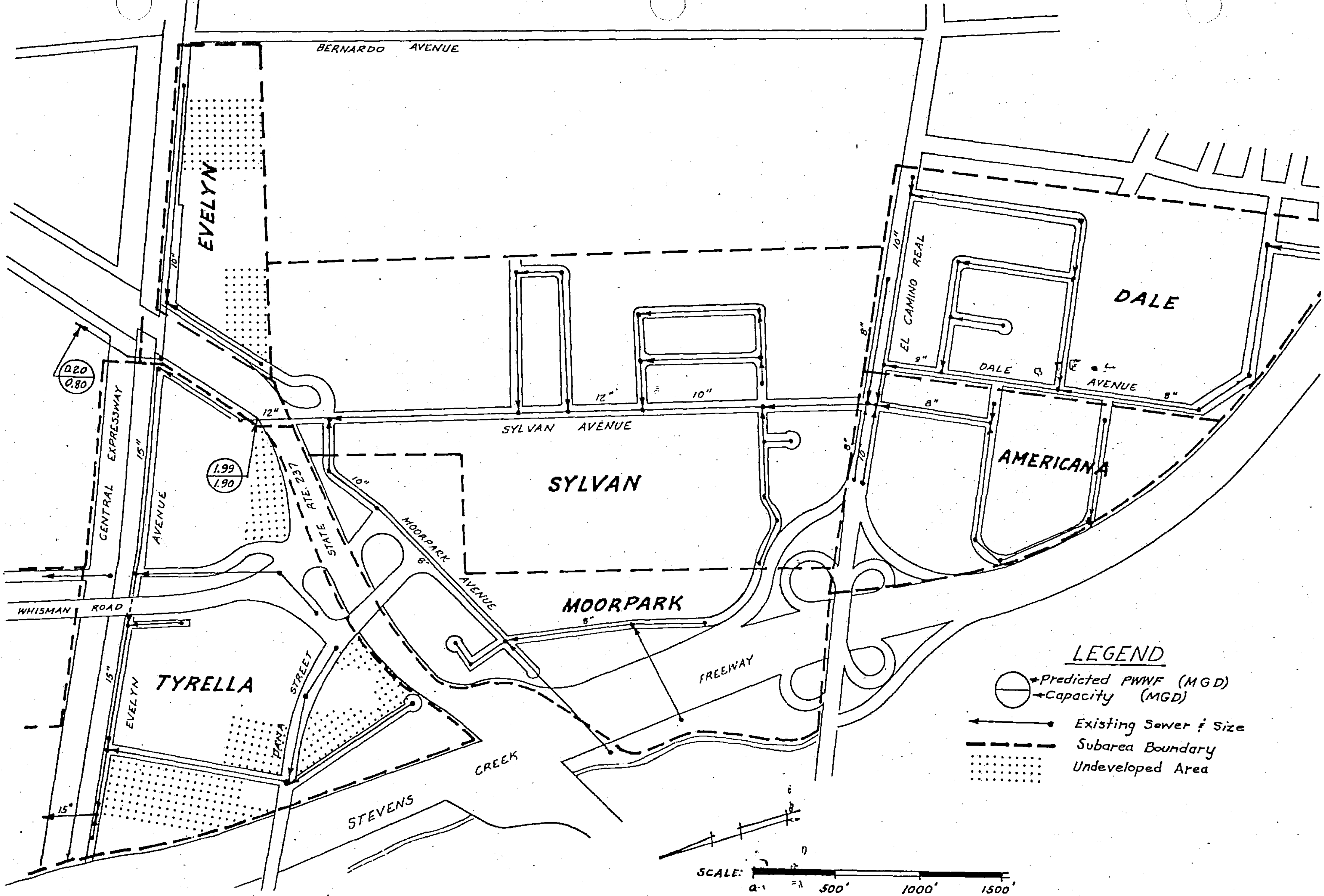


FIG. 3-1A
PREDICTED WASTEWATER FLOW FOR
FUTURE DEVELOPMENT LIMITED TO 6000 GPAD

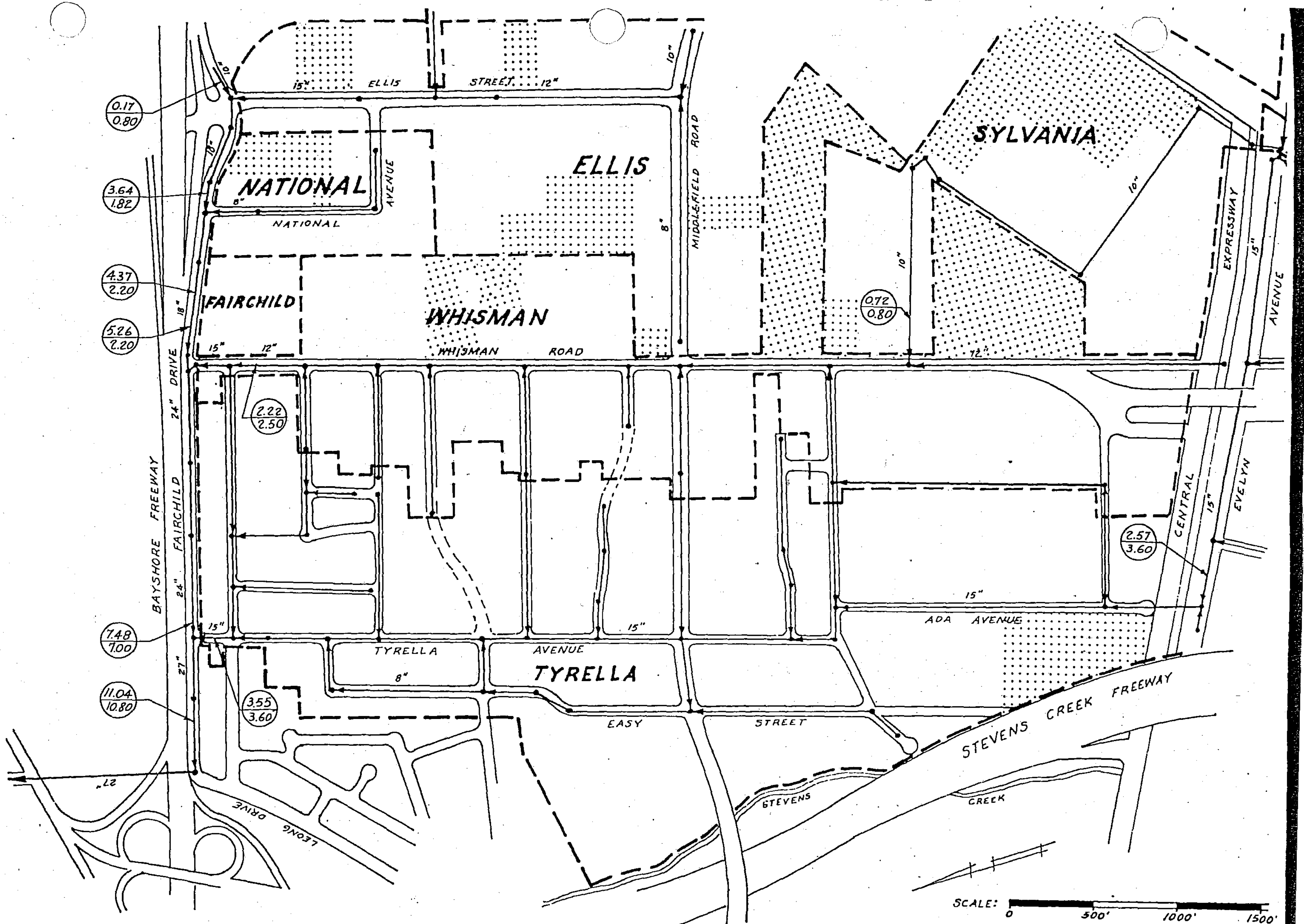


FIG. 3-1B
PREDICTED WASTEWATER FLOW FOR
FUTURE DEVELOPMENT LIMITED TO 6000 GPAD

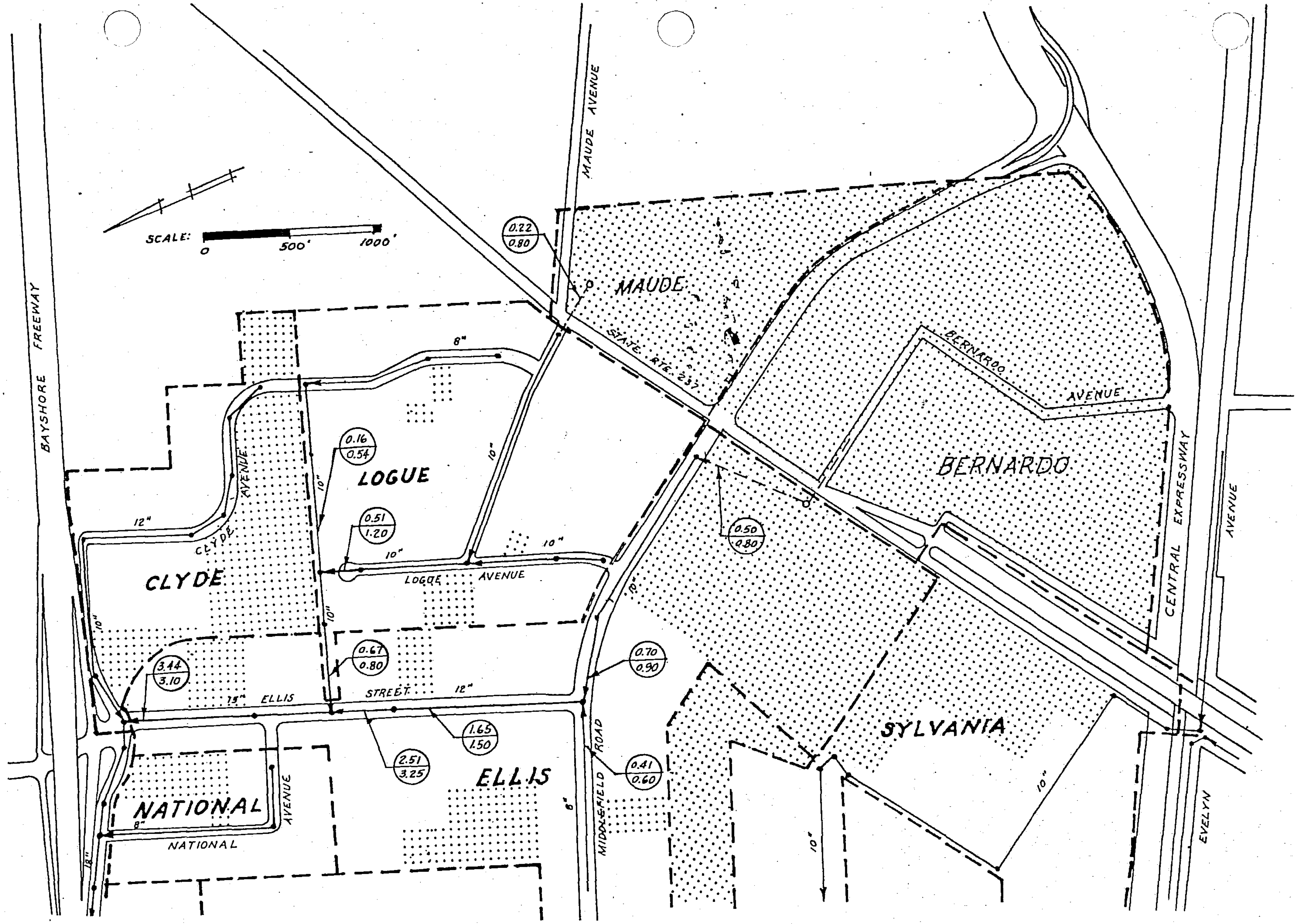


FIG. 3-1C
PREDICTED WASTEWATER FLOW FOR
FUTURE DEVELOPMENT LIMITED TO 6000 GPAD

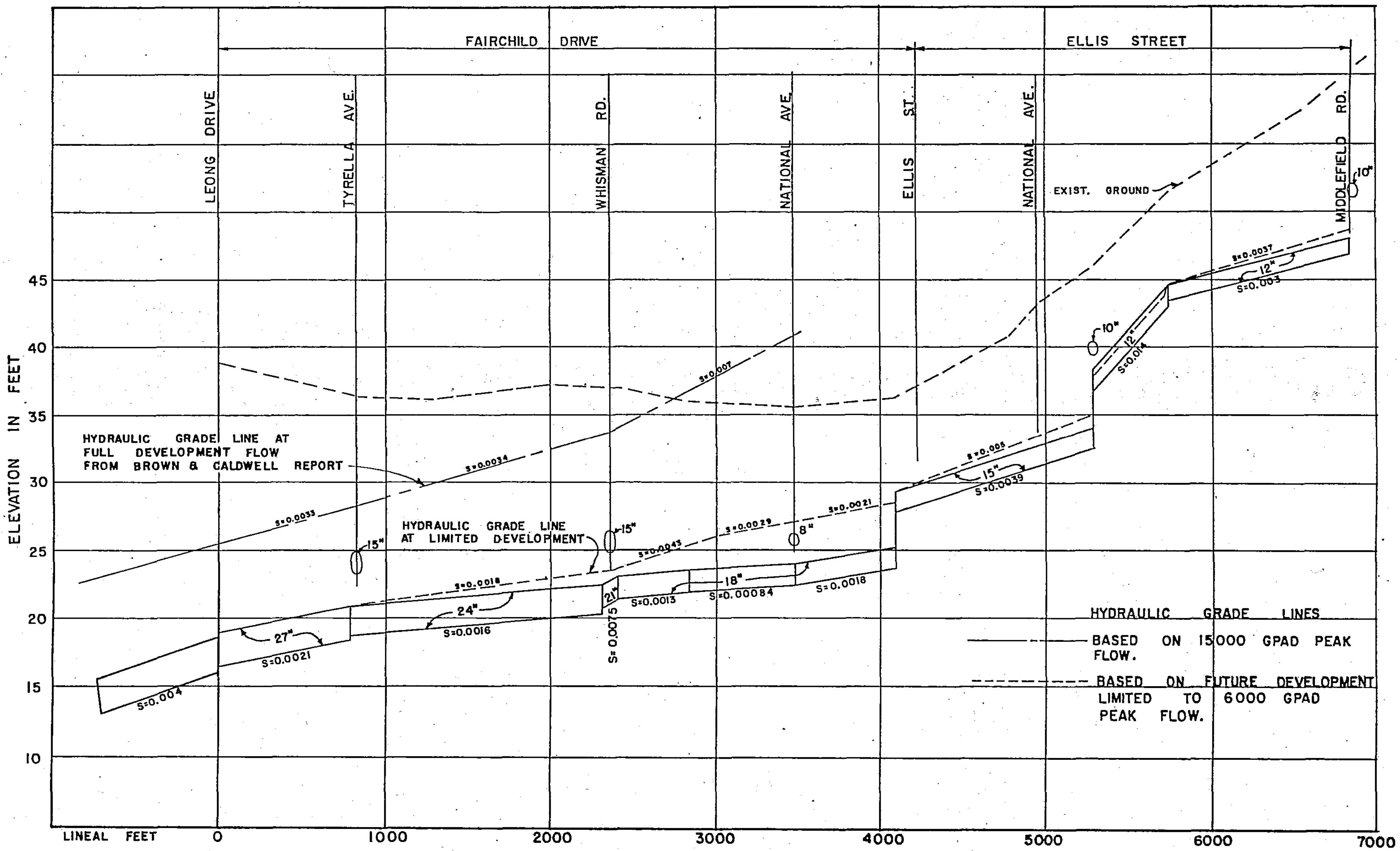


FIG. 3-2
PEAK FLOW HYDRAULIC—PROFILE EAST TRUNK SEWER

CHAPTER 4

ALTERNATIVE SOLUTIONS

Alternative Solutions

To arrive at the best solution to the problem, several alternative solutions have been considered in detail. These alternative solutions may be grouped into four basic alternates, as follows:

ALTERNATE A - Construct new sewers

ALTERNATE B - Allow sewers to surcharge

ALTERNATE C - Limit sewage flows

ALTERNATE D - Rearrange sewer system

In this chapter the estimated cost of each alternate will be given as well as advantages and disadvantages of each alternate.

ALTERNATE A - Construct New Sewers

This alternate is the plan recommended in the Brown and Caldwell report. It is based on an average sewage flow of 5,000 gallons per acre per day (g.p.a.d.) and a peak sewage flow of 15,000 g.p.a.d. The plan is for full industrial development. This alternate includes the following:

1. Construct a 12-inch sewer from the S.P. tracks at Sylvania to Whisman Road to intercept the sewage flow from south of the Central Expressway and from Sylvania

440-ft. @ $20,000

2. Construct a 12-inch sewer on Whisman Road from Sherland Avenue to Evandale Avenue to serve as a relief sewer for parallel existing 12-inch trunk sewer — 1,500-ft. @ $50,000

3. Construct a 21-inch sewer on Fairchild Drive from Ellis Avenue to the East Trunk sewer at Leong Drive to serve as a relief sewer for the parallel existing 18" to 27" trunk sewer — 4,100-ft. @ $260,000

4. Construct a 12-inch sewer from Logue Avenue to Fairchild Drive to serve as a relief sewer for 15-inch existing sewer in Ellis Avenue and for 10-inch existing sewer from Logue Avenue to Ellis Avenue — 2,100-ft. @ $85,000

5. Construct a 12-inch sewer on Maude Avenue and Logue Avenue to connect currently undeveloped area east of Route 237 — 1,700-ft. @ $60,000

6. Construct 12-inch and 15-inch sewers to serve currently undeveloped area east of Route 237 — 2,900-ft. @ $165,000

TOTAL CONSTRUCTION COST: $640,000

This plan has the advantage that it will solve all expected problems with a basically permanent trouble-free solution that has sufficient flexibility to serve a wide variety of industrial development.

It has the disadvantage that it will permit industrial growth which could generate enough sewage to cause Mountain View's current capacity rights to be exceeded at the Regional Water Quality Control Plant in Palo Alto. It has the added disadvantage that it is the most costly alternate and does not attempt to discourage present wasteful water use practices.

ALTERNATE B - Allow Existing Sewers to Surcharge

Sewer surcharging occurs when more sewage is discharged to the sewer than the sewer was designed to handle. In the previous chapter, Figure 3-2 shows the amount of surcharging which can be expected under future flow conditions.

In this alternate, several sewers would be allowed to surcharge daily at peak flow periods. This alternate, therefore, avoids construction of several of the sewers in Alternate A. Those sewers which can be surcharged at peak future flows without overflowing or backing up into lateral sewers are:

1. Fairchild Drive from Whisman Road to Leong Drive
2. Maude Avenue from Route 237 to Logue Avenue
3. Logue Avenue from Maude to Hetch-Hetchy R.O.W.
4. Whisman Road

Surcharging of these mains would be only a partial solution, however, if industrial development reaches the level predicted by Brown and Caldwell.

New sewers would still be needed to serve the area east of Route 237, and relief sewers would be needed where surcharging cannot be tolerated. The timing of sewer construction would depend on actual development, but sewers would be designed for flows recommended by Brown and Caldwell. This alternate includes the following:

1. Construct 12-inch and 15-inch sewers to serve currently undeveloped area east of Route 237	2,900-ft. @ $165,000
2. Construct a 15-inch sewer from Logue Avenue to Fairchild Drive to serve as a relief sewer for 15-inch existing sewer in Ellis Avenue	2,100-ft. @ $ 85,000
3. Construct a 21-inch sewer on Fairchild Drive from Ellis Avenue to Whisman Road to serve as a relief sewer for the parallel existing 18-inch trunk sewer	1,800-ft. @ $115,000
TOTAL CONSTRUCTION COST:	$365,000

This alternate has the same advantages and disadvantages as Alternate A except that the plan is less trouble-free because of the surcharging permitted and less flexible because it uses up most of the reserve capacity of several existing sewers.

It has the added advantage of lower overall cost than Alternate A.

ALTERNATE C - Limit Sewage Flows

The existing industrial area sewerage system was designed on the basis of 6,000 g.p.a.d. peak sewage flow. In this alternate future extensions of the system would be designed on the same basis and future industrial development would be restricted in their water use to this rate of discharge. Present industries that exceed this rate of discharge would likewise be required by ordinance to reduce their discharges to stay within design limits.

This alternate includes the following:

1. Construct 10-inch sewers to serve the currently undeveloped area east of Route 237	2,900-ft. @ $ 90,000
2. Limit by ordinance to 6,000 g.p.a.d. all industrial discharges to the sewerage system	
3. Staff work to carry out the program	$ 2,000/Yr.
TOTAL CONSTRUCTION COST:	$ 90,000

The major advantage of this alternate is low cost to the City. Item 1 would at least partially be paid for by developers. Item 2 could cost about $500,000 for in-plant industrial modifications to recycle water. This would be offset by reductions in sewer charges of about $100,000/yr. This loss in revenue to the City would eventually be offset by further industrial development up to the limit of system capability and would also be offset by lower costs for sewage treatment.

Additional advantages include forestalling the need of purchase of additional capacity at the Regional Water Quality Control Plant, elimination of current wasteful water use practices, and elimination of the disruptive aspects of sewer construction.

A disadvantage of this plan would be the increased staff time required to monitor industrial discharges and to enforce peak flow limits. This could cost about $2,000 per year.

ALTERNATE D - Rearrange Sewer System

Occasionally it is possible to use an existing sewer system in a different way to achieve satisfactory solutions to localized problems. This alternate considers the following possibilities:

1. Divert industrial flow to residential sewers.
   Since residential peak flows on Tyrella Avenue occur a few hours before the industrial peak flow, the residential sewer has capacity to receive these flows during the daytime. Diversion could be accomplished by installing a dam in the Whisman Road sewer. This would intentionally surcharge the sewer and force the flow across to Tyrella Avenue. This diversion could be accomplished for about $ 5,000

2. In the entire industrial area, a considerable amount of water is used for cooling purposes. This water is either evaporated or discharged to the sewer. It

is pure water and needs no treatment, therefore it could be discharged to the storm drainage system rather than to the sanitary sewer. Since most cooling units are located near roof drains, the cost of transfer would be very small and would be accomplished by the industries.

3. Many industries produce high purity process water by a treatment process known as reverse osmosis. This method employes a semi-permeable membrane which rejects dissolved minerals and allows only demineralized water to pass through. About 30 percent of the water supplied to the unit is used to wash the rejected minerals from the membrane and is discharged to the sanitary sewer. This water discharge is still a high quality water even though more concentrated in minerals than the water supply. This water should be suitable for discharge to the storm drainage system. The connections would be installed by industry.

4. Construct 10-inch sewers to serve the currently undeveloped area east of Route 237

2,900-ft. @ $ 90,000

5. Construct a 21-inch sewer on Fairchild Drive from Ellis Avenue to Whisman Road to serve as a relief sewer for the parallel existing 18-inch trunk sewer 1,800-ft. @ $115,000

6. Staff work to carry out the program $ 2,000/Yr.

TOTAL CONSTRUCTION COST: $205,000

The major advantage of this alternate is eliminating pure water discharges to the sewerage system. While these flows produce revenue, they use up trunk sewer and treatment plant capacity and cause a wasteful use of treatment chemicals and power.

The amount of flow removed, however, would not be sufficient to prevent overloading of the Fairchild Drive trunk sewer and a relief sewer would be required.

The major disadvantage would be the introduction of potentially hazardous industrial wastewater into a residential sewer.

CHAPTER 5

RECOMMENDED PLAN

The recommended plan for the solution of the problem is basically that of Alternate C described in the previous chapter with some elements of Alternate D included. The plan includes the following elements:

1. As industrial development continues, extend the East Trunk sewer system using the same design basis as the existing system. Estimated cost for extensions east of Route 237 is \$90,000 a large part of which will be financed and constructed by developers.

2. Review all proposals for industrial development to assure that new industries will not discharge wastewaters in excess of system capabilities.

3. Encourage existing industries to reduce existing peak flow-rates by scheduling peak discharges, by permitting discharges of unpolluted wastewater to storm drains, and by eliminating wasteful water uses. Authority for this is covered by the current Industrial Waste Ordinance.

4. Monitor wastewater flows at key points in the sewer system to assure that sewers are not being unduly overloaded. Costs associated with this work are estimated at \$2,000 per year.

This work would be an extension of present Public Works programs and no additional staff is requested.

The recommended plan is in essence one of bringing flows into line with the original system design rather than to redesign the system to meet all possible future sewage flows.

The plan takes into account the fact that the major industries in the study area now discharge at considerably higher rates than the system was designed to handle. Calculations show that these flows, though excessive, can be tolerated primarily due to their downstream locations in the system as long as future development is limited to discharges of 6,000 g.p.a.d.

The existing system has considerable reserve capacity because the East Trunk sewer is deep and can tolerate a large surcharge without overflowing. Reductions in planned downstream development in the north of Bayshore area will also eliminate the possibility of downstream surcharging. Therefore, any surcharging in the study area will be locally caused.

To keep surcharging to a minimum industries will be encouraged to reduce their sewage discharges through water conservation and reuse techniques. Industries that discharge at greater than design flowrates may also be required to pay additional fees for excessive flows, to transfer clean water discharges from the sanitary sewer system to the storm drainage system, and may be required to limit their peak discharges to specified hours when the system is more capable of handling excess flows. The present industrial waste ordinance provides for all of these conditions.

To serve the unsewered industrial area east of Route 237, it will be necessary to install new sewers which connect to the existing East Trunk sewer system. It is recommended that these sewers should not have greater capacity than the system they drain to. The sewers, therefore, should be based on a peak dry weather flow of 6,000 g.p.a.d.

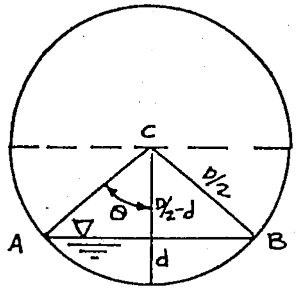
The recommended plan, therefore, involves low cost to the City and should involve costs to industry only to the extent that is reasonable in eliminating wasteful uses of water.

As part of this study, a preliminary evaluation of existing sewer service charges was made. From this study it was determined that in general, industries have been paying for their share of sewerage costs. Future regional treatment and disposal works, however, are expected to cause substantial increases in charges to industry. Rising costs for operation and maintenance of the existing regional plant will also cause a general rate increase. A separate report on this subject will be prepared in the near future.

APPENDIX

Determination of Slope

When Depth of Flow is less than Half Full



$$\text{Area of circle segment } ACB = \frac{\pi D^2}{4}\left(\frac{2\theta}{360}\right)$$

$$\theta = \cos^{-1}\left(\frac{D/2 - d}{D/2}\right)$$

$$\therefore \text{Area of circle segment } ACB = \frac{\pi D^2}{720}\cos^{-1}\left(\frac{D/2-d}{D/2}\right)$$

$$\text{Area of triangle } ABC = (D/2 - d)\sqrt{(D/2)^2 - (D/2-d)^2}$$

$$\therefore \text{Area of water cross-section} = \frac{\pi D^2}{720}\cos^{-1}\left(\frac{D/2-d}{D/2}\right) - (D/2-d)\sqrt{(D/2)^2-(D/2-d)^2}$$

$$\text{Wetted Perimeter of arc } AB = \frac{\pi D 2\theta}{360} = \frac{\pi D}{180}\cos^{-1}\left(\frac{D/2-d}{D/2}\right)$$

$$\text{Hydraulic Radius } (R) = \frac{A}{W.P.} = \frac{\frac{\pi D^2}{720}\cos^{-1}\left(\frac{D/2-d}{D/2}\right) - (D/2-d)\sqrt{(D/2)^2-(D/2-d)^2}}{\frac{\pi D}{180}\cos^{-1}\left(\frac{D/2-d}{D/2}\right)}$$

Use Mannings Formula $\quad V = \frac{1.486}{n} R^{2/3} S^{1/2}$

$$\therefore \quad S = \left(\frac{V \cdot n}{1.486 \cdot R^{2/3}}\right)^2$$

$$\boxed{\text{or} \quad S = \left\{\frac{V \cdot n}{1.486\left[\frac{\frac{\pi D^2}{720}\cos^{-1}\left(\frac{D/2-d}{D/2}\right) - (D/2-d)\sqrt{(D/2)^2-(D/2-d)^2}}{\frac{\pi D}{180}\cos^{-1}\left(\frac{D/2-d}{D/2}\right)}\right]^{2/3}}\right\}^2}$$

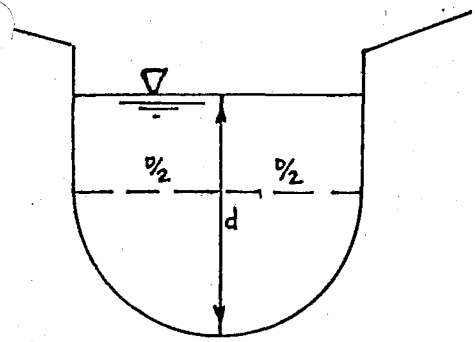
<u>Example: M.H.#1</u> $\quad d = 1.1'$ (Measured) $\quad V = 4.1546$ f.p.s. (Measured) $\quad D = 2.25'$ $\quad n = .011$

$$S = \left\{\frac{4.1546 \times .011}{1.486\left[\frac{\frac{\pi 2.25^2}{720}\cos^{-1}\left(\frac{1.125-1.1}{1.125}\right) - (1.125-1.1)\sqrt{1.125^2 - (1.125-1.1)^2}}{\frac{\pi D}{180}\cos^{-1}\left(\frac{1.125-1.1}{1.125}\right)}\right]^{2/3}}\right\}^2$$

$$S = \left\{\frac{.0457}{1.486\left(\frac{1.9599 - .02812}{3.4843}\right)^{2/3}}\right\}^2 = \left(\frac{.0457}{1.0029}\right)^2 = \underline{\underline{.0021}}$$

Determination of Slope

When Depth of Flow is more than Half Full



Area of water cross-section $= (d - D/2)D + \frac{\pi D^2}{4 \cdot 2}$

Wetted Perimeter $= \frac{\pi D}{2} + (d - D/2)2$

Hydraulic Radius $(R) = \frac{A}{W.P.} = \dfrac{(d - D/2)D + \frac{\pi D^2}{8}}{\frac{\pi D}{2} + (d - D/2)2}$

Use Mannings Formula $\quad V = \frac{1.486}{n} R^{2/3} S^{1/2}$

$$\therefore \quad S = \left(\frac{V \cdot n}{1.486 \cdot R^{2/3}} \right)^2$$

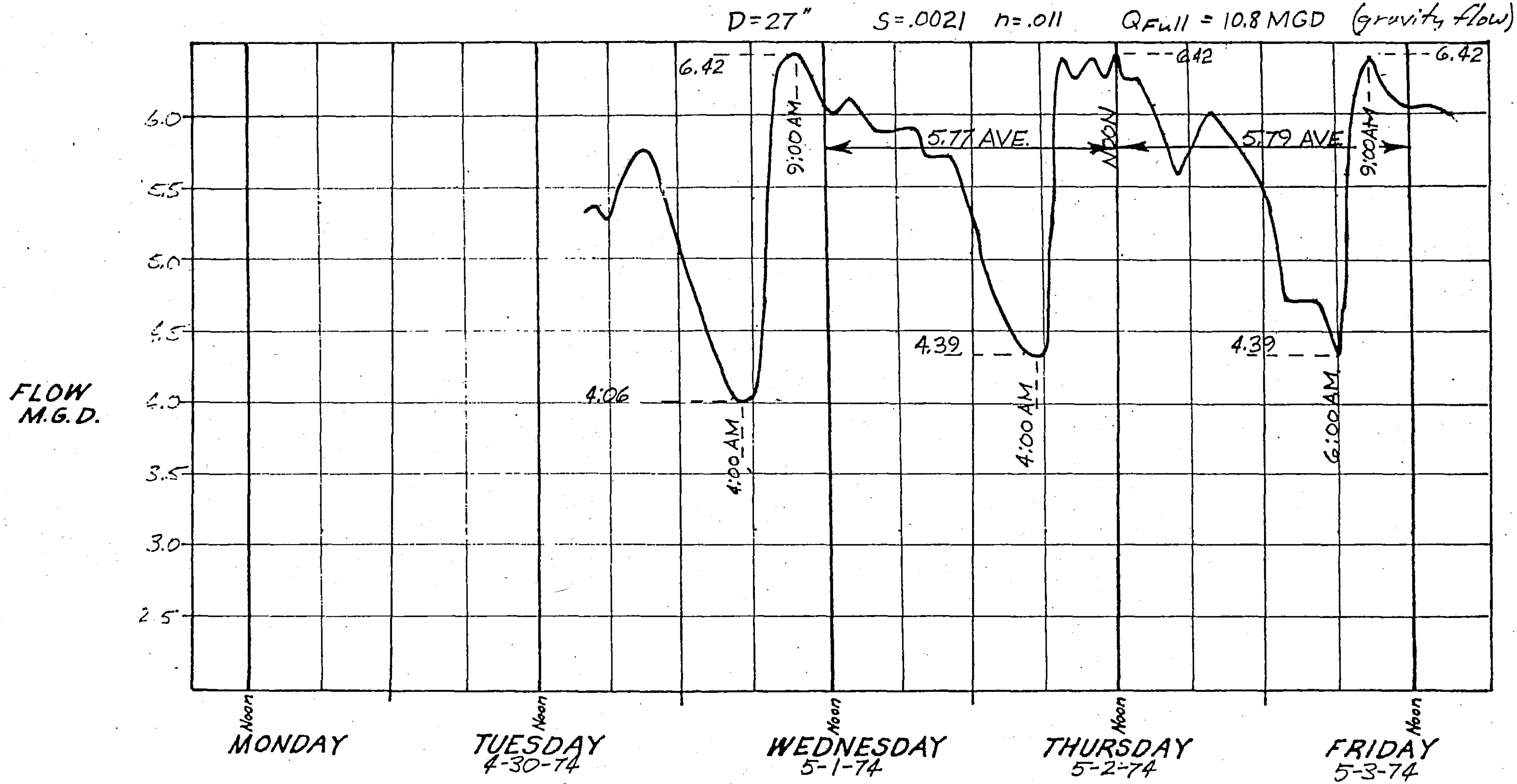
$$\boxed{S = \left\{ \frac{V \cdot n}{1.486 \left[\dfrac{(d - D/2)D + \frac{\pi D^2}{8}}{\frac{\pi D}{2} + (d - D/2)2} \right]^{2/3}} \right\}^2}$$

<u>Example: M.H. #5</u> $\quad$ d = 1.08' (Measured) $\quad$ V = 3.5643 (Measured) $\quad$ D = 2.0' $\quad$ n = .011

$$S = \left\{ \frac{3.5643 \times .011}{1.486 \left[\dfrac{(1.08 - 1)2 + \frac{\pi 4}{8}}{\frac{\pi 2}{2} + (1.08 - 1)2} \right]^{2/3}} \right\}^2$$

$$S = \left\{ \frac{.0392}{1.486 \left(\frac{1.3394}{3.3082} \right)^{2/3}} \right\}^2 = \left(\frac{0.0392}{0.9673} \right)^2 = .0016$$

CITY OF MOUNTAIN VIEW
EAST TRUNK SANITARY SEWER STUDY

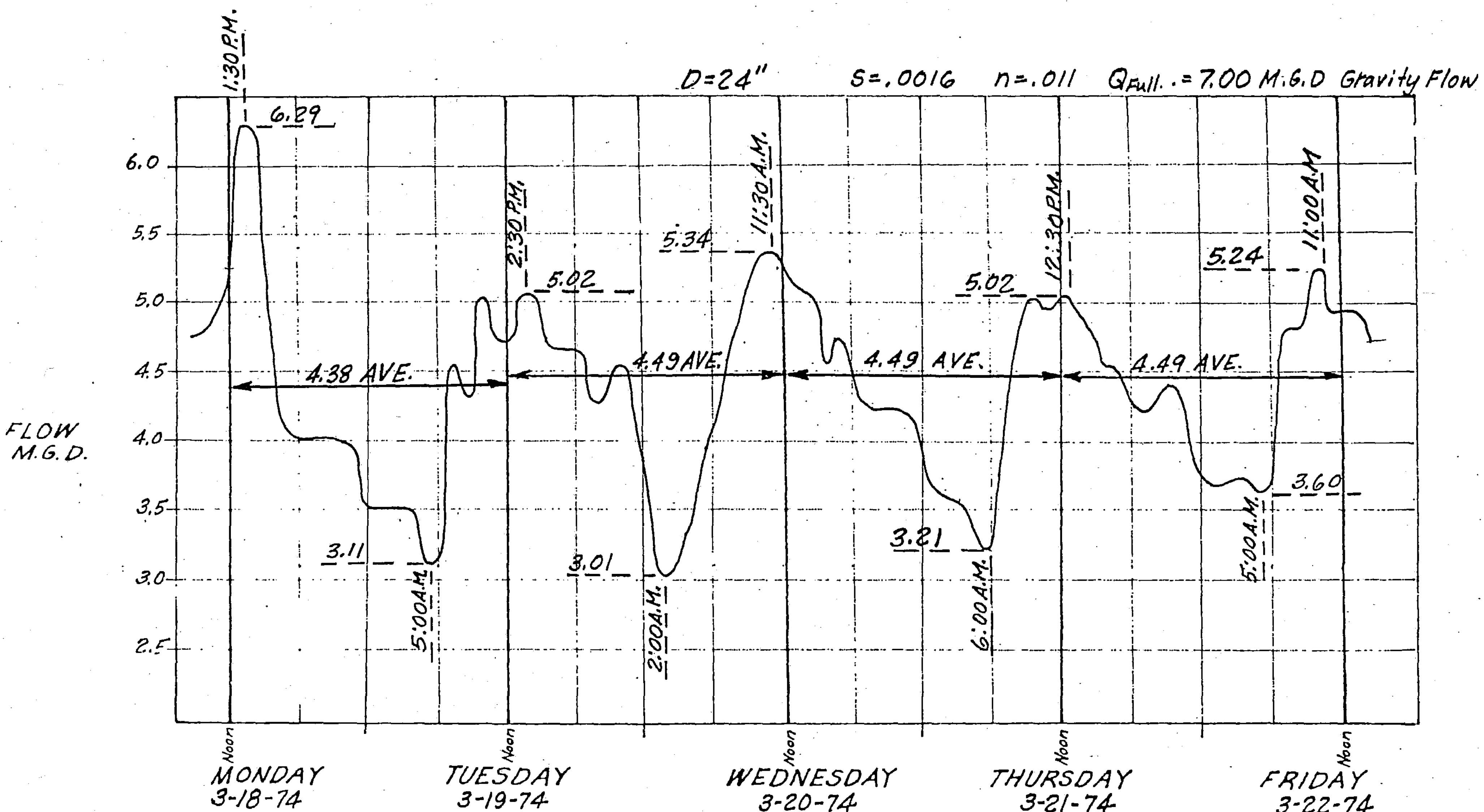


Ave. 2 Day Flow 5.78 MGD
MAX. Peaking Factor = 1.11

FAIRCHILD DR. M.H. # 1

CITY OF MOUNTAIN VIEW

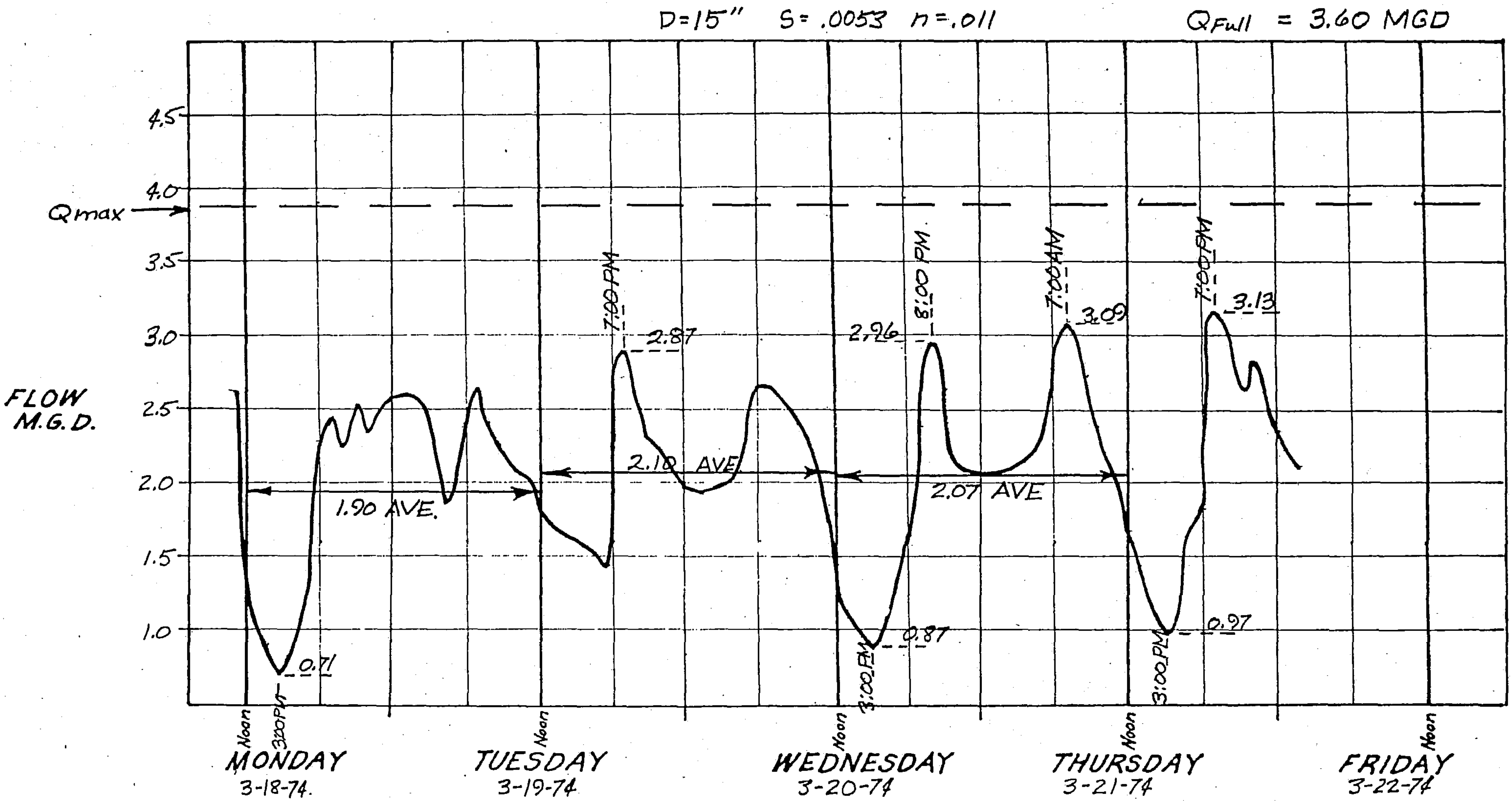
EAST TRUNK SANITARY SEWER STUDY



AVE. 4-DAY FLOW 4.46 M.G.D.
MAX. PEAKING FACTOR = 1.41

FAIRCHILD DR. M.H. #2

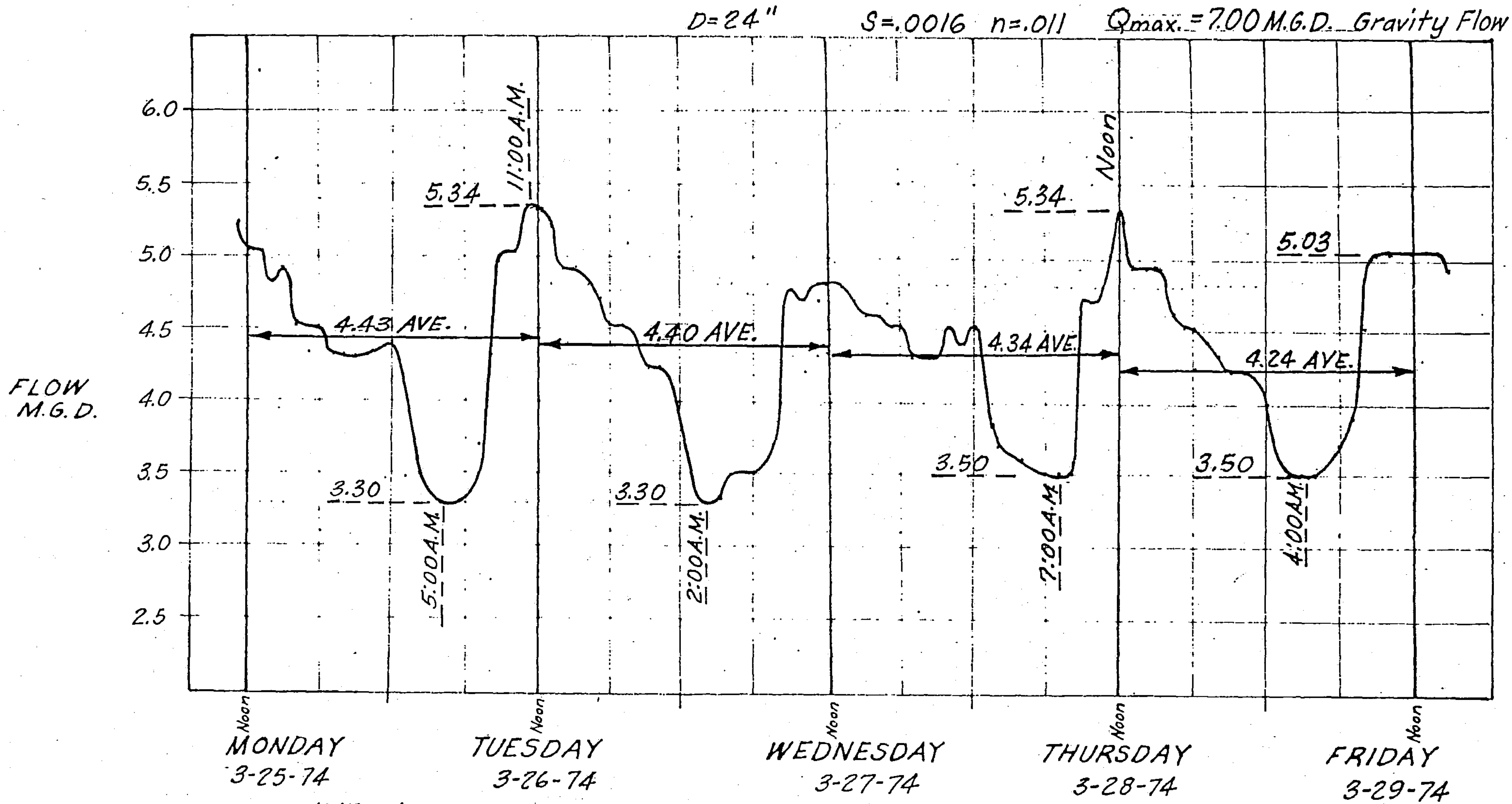
CITY OF MOUNTAIN VIEW
EAST TRUNK SANITARY SEWER STUDY



AVE. 3 DAY FLOW 2.02 MGD
MAX PEAKING FACTOR 1.56

TYRELLA AVE. M.H. # 3

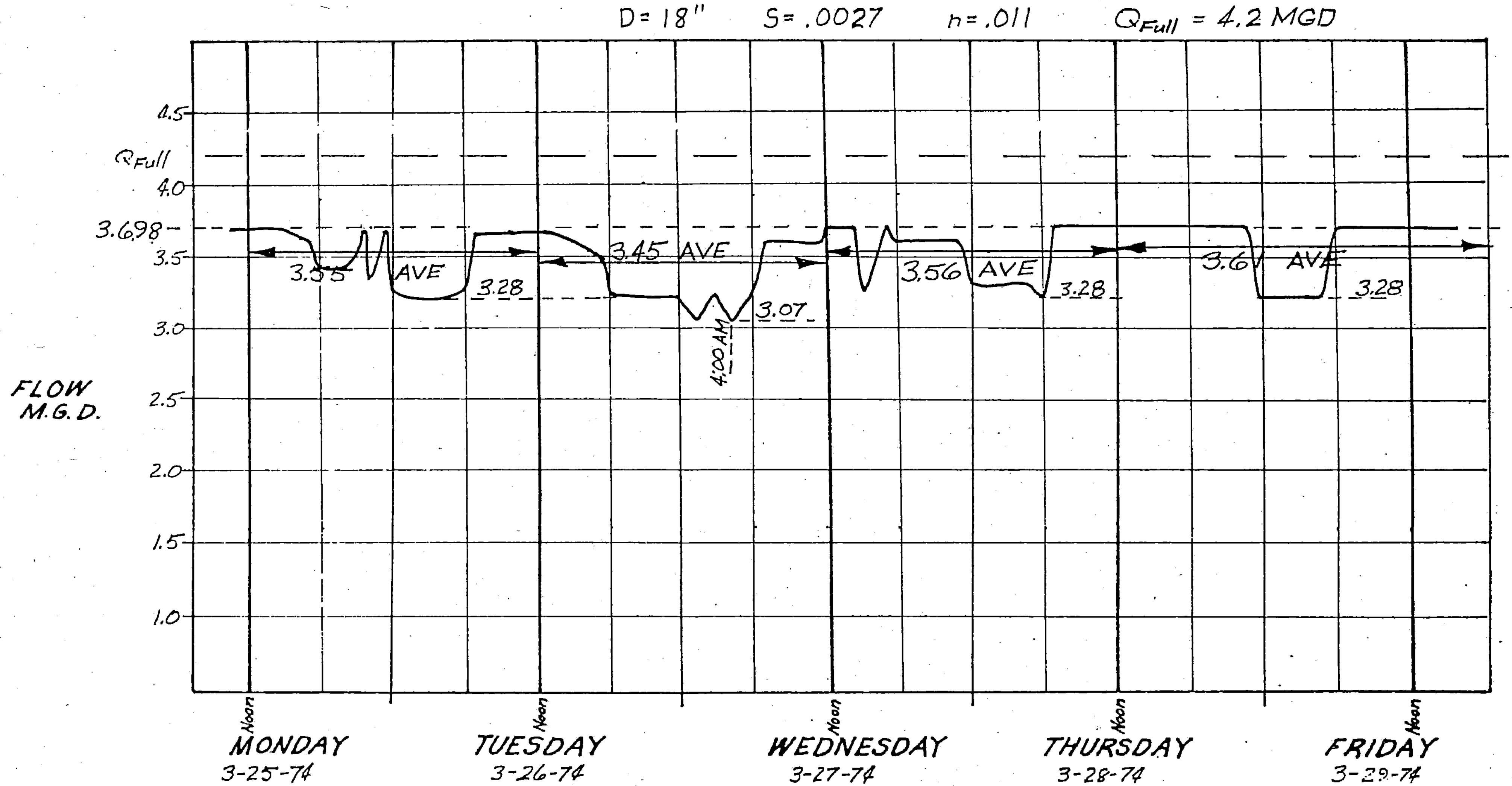
CITY OF MOUNTAIN VIEW
EAST TRUNK SANITARY SEWER STUDY



AVE. 4-DAY FLOW 4.35 M.G.D.
MAX. PEAKING FACTOR = 1.23

FAIRCHILD DR. M.H. #5

CITY OF MOUNTAIN VIEW
EAST TRUNK SANITARY SEWER STUDY

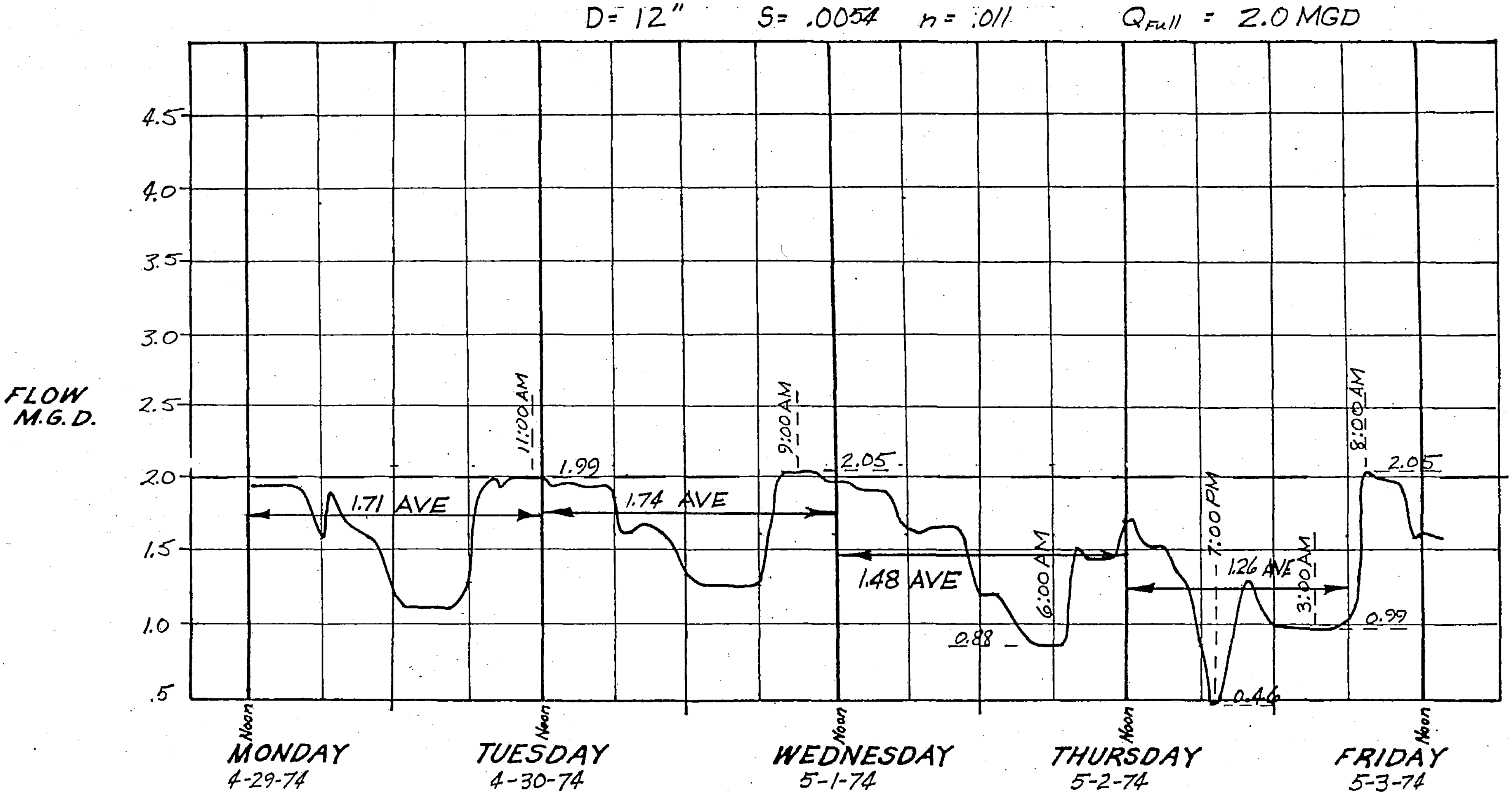


AVE 4 DAY FLOW = 3.46 MGD
MAX PEAKING FACTOR = 1.07

FAIRCHILD DR.

M.H. # 7

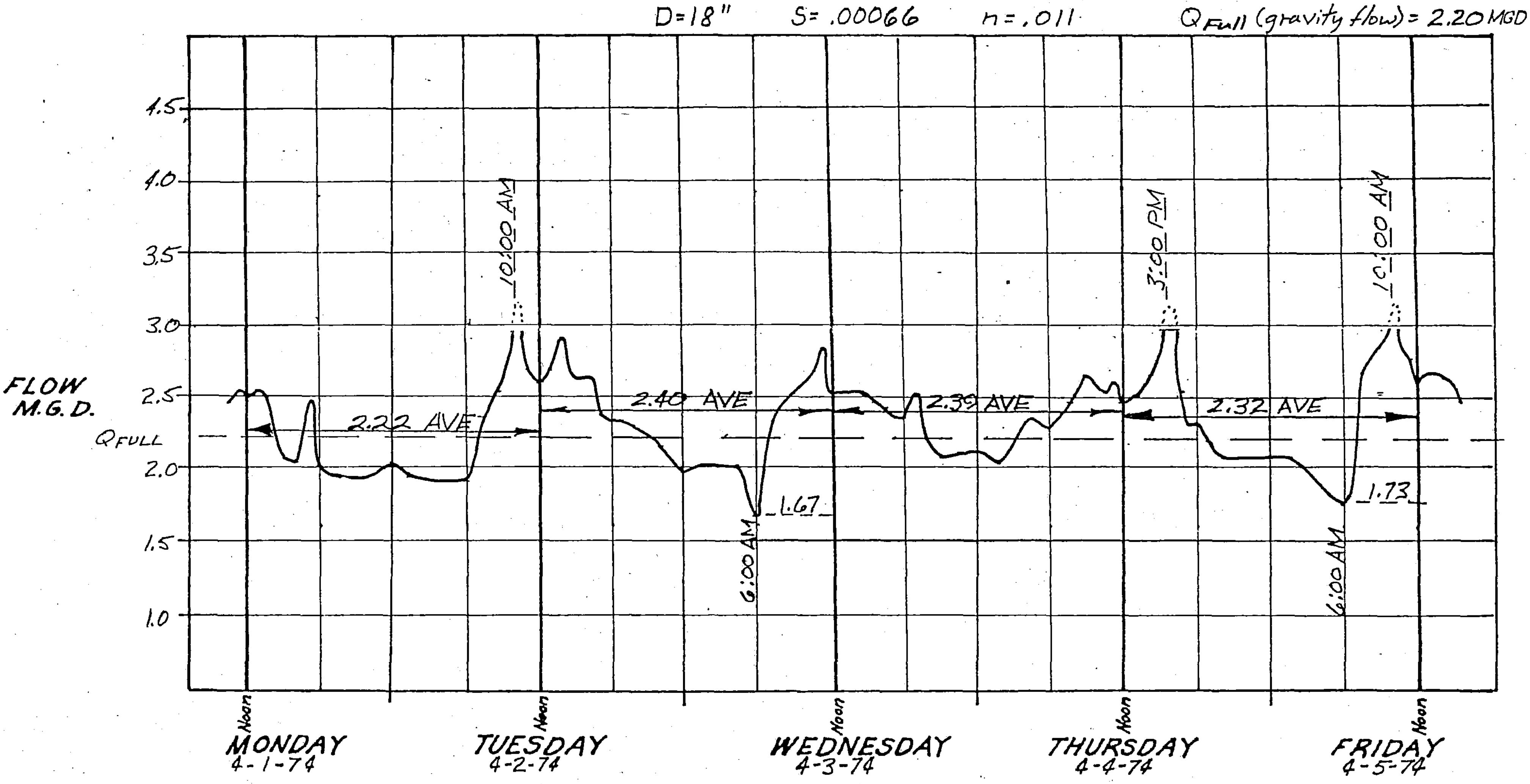
CITY OF MOUNTAIN VIEW
EAST TRUNK SANITARY SEWER STUDY



AVE 4 DAY FLOW = 1.55 MGD
MAX PEAKING FACTOR = 1.32

WHISMAN RD. M.H. # 8

CITY OF MOUNTAIN VIEW
EAST TRUNK SANITARY SEWER STUDY

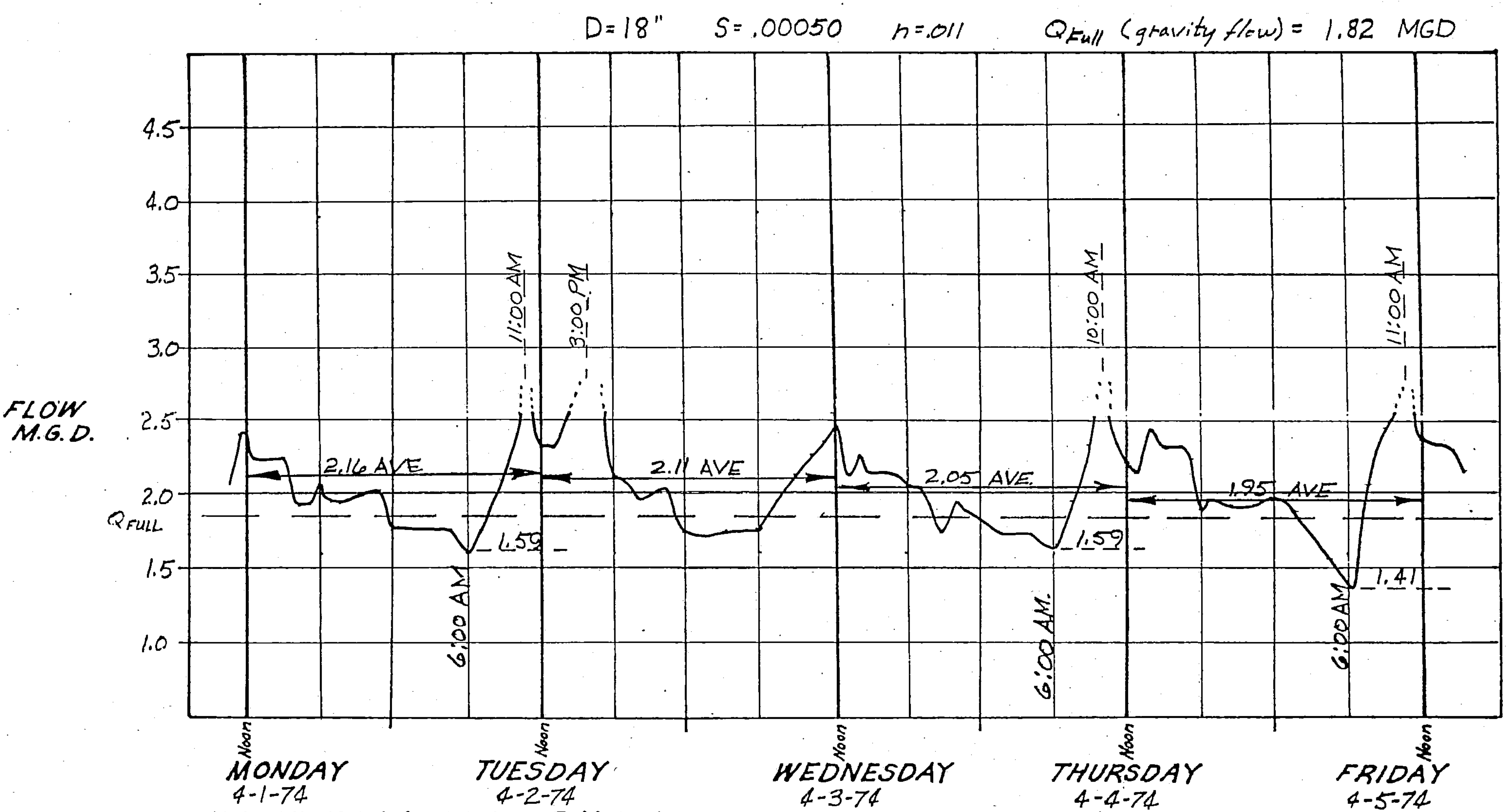


AVE. FOUR DAY FLOW = 2.33 MGD
MAX PEAKING FACTOR ?

FAIRCHILD DR.

M.H. # 9

CITY OF MOUNTAIN VIEW
EAST TRUNK SANITARY SEWER STUDY

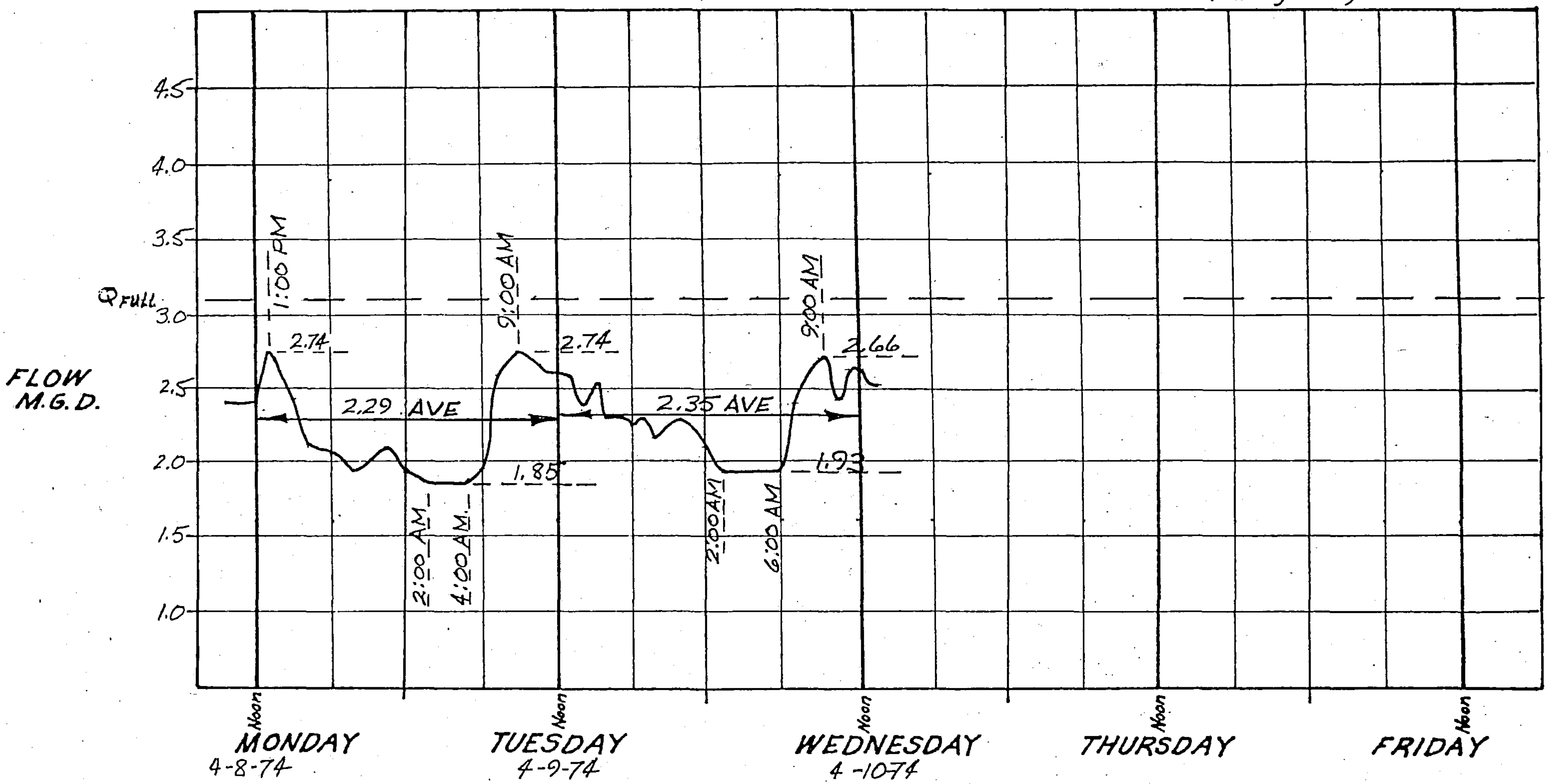


AVE. FOUR DAY FLOW = 2.16 MGD
MAX. PEAKING FACTOR = ?

FAIRCHILD DR. M.H. #10

CITY OF MOUNTAIN VIEW
EAST TRUNK SANITARY SEWER STUDY

D= 15" S= .0039 n=.011 Q_{Full} (gravity) = 3.1 MGD

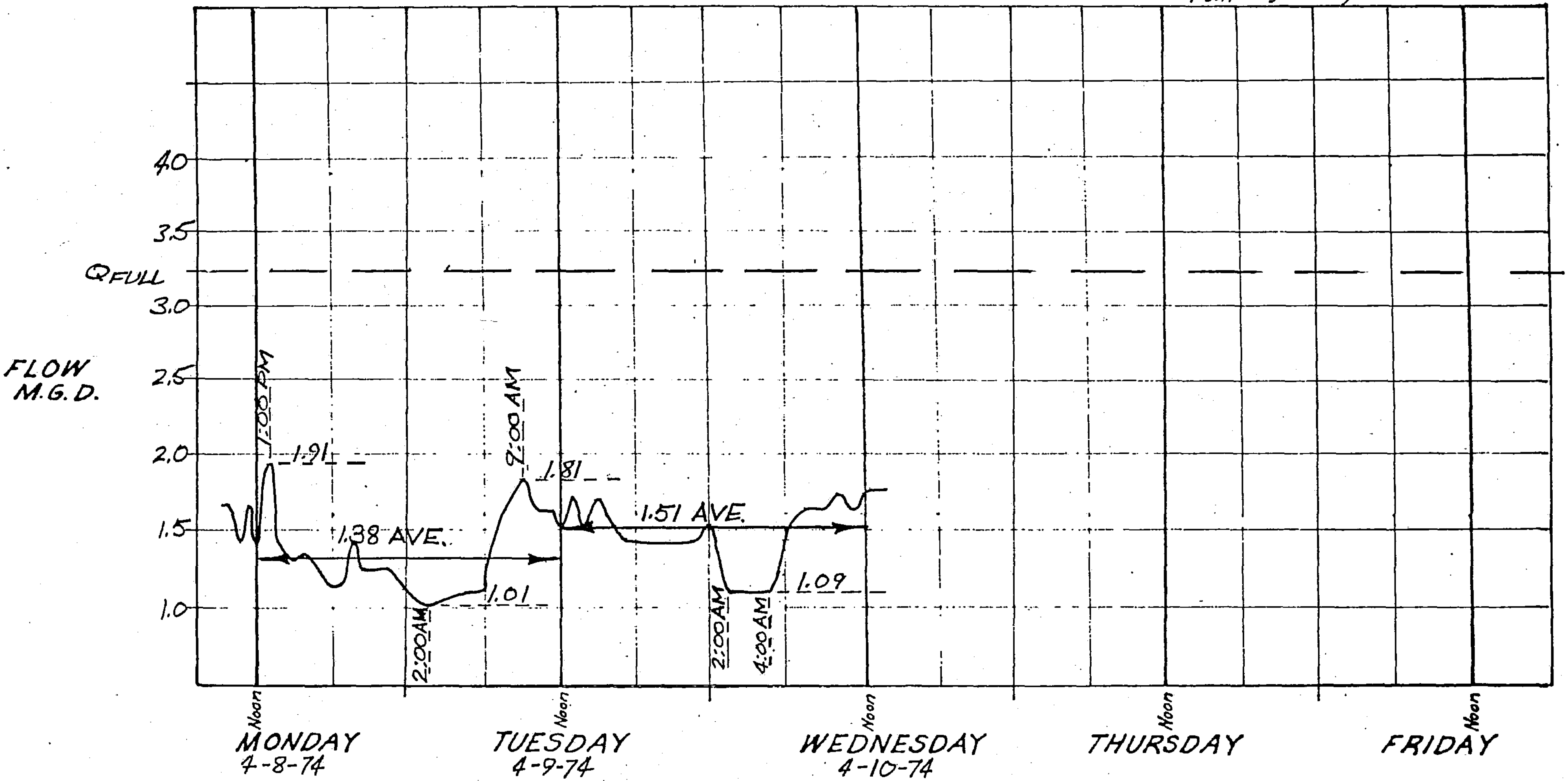


AVE. TWO DAY FLOW = 2.32 MGD
MAX PEAKING FACTOR = 1.18

ELLIS ST. M.H. # 13

CITY OF MOUNTAIN VIEW
EAST TRUNK SANITARY SEWER STUDY

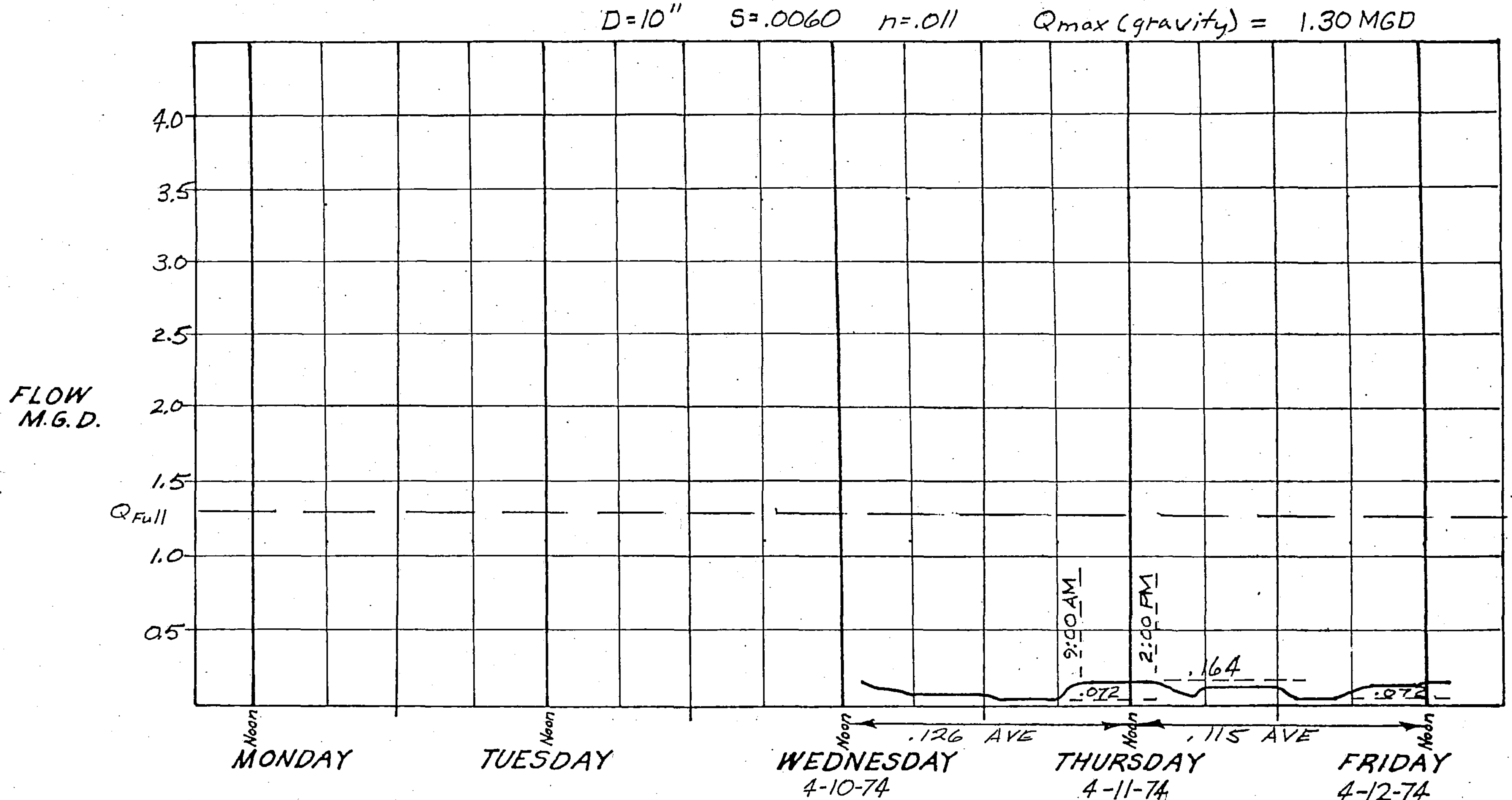
D= 12"   S=.014   n=.011 Q_{Full} (gravity) = 3.25 MGD



AVE. TWO DAY FLOW = 1.45 MGD
MAX PEAKING FACTOR = 1.32

ELLIS ST.   M.H. #14

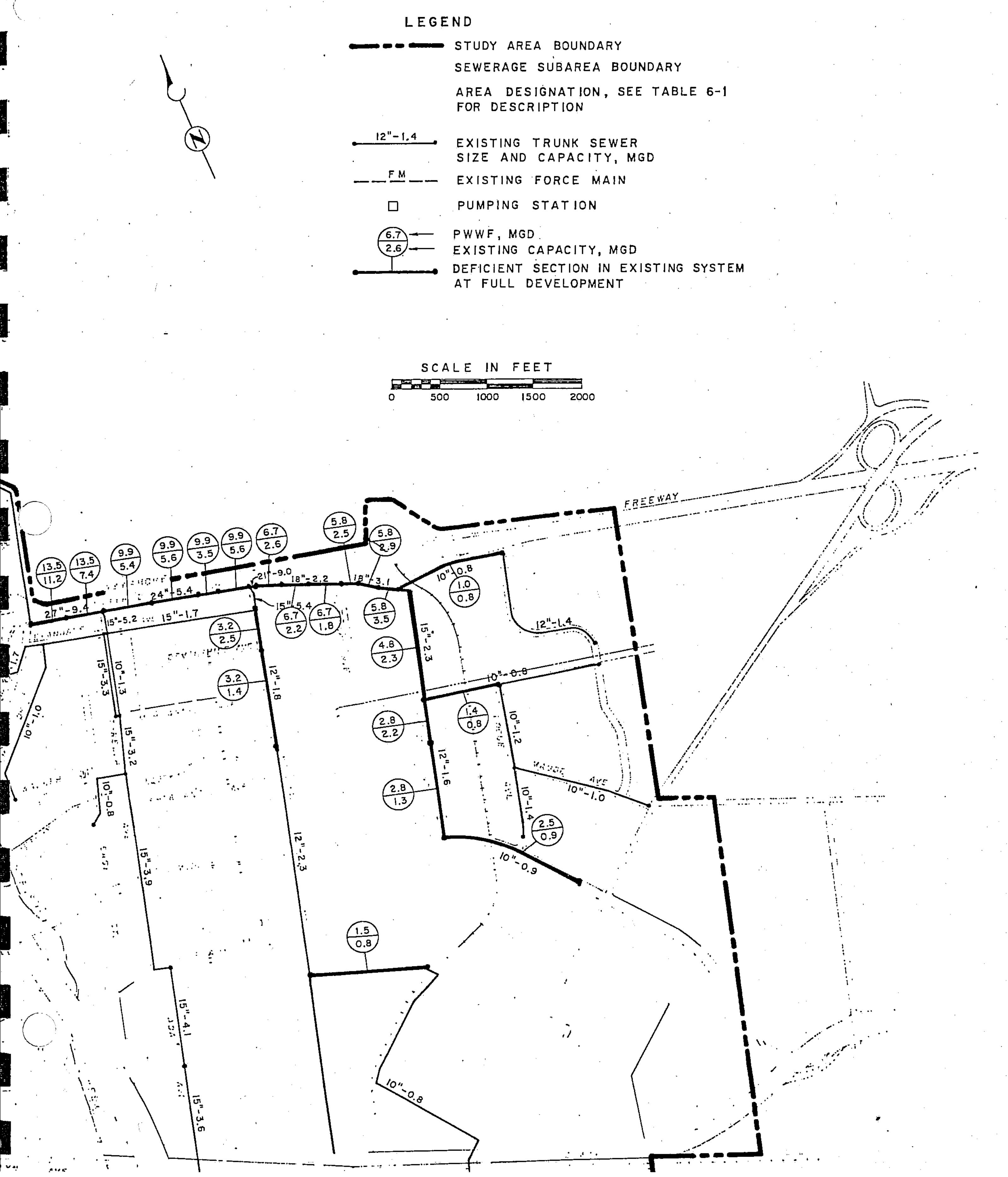
CITY OF MOUNTAIN VIEW
EAST TRUNK SANITARY SEWER STUDY

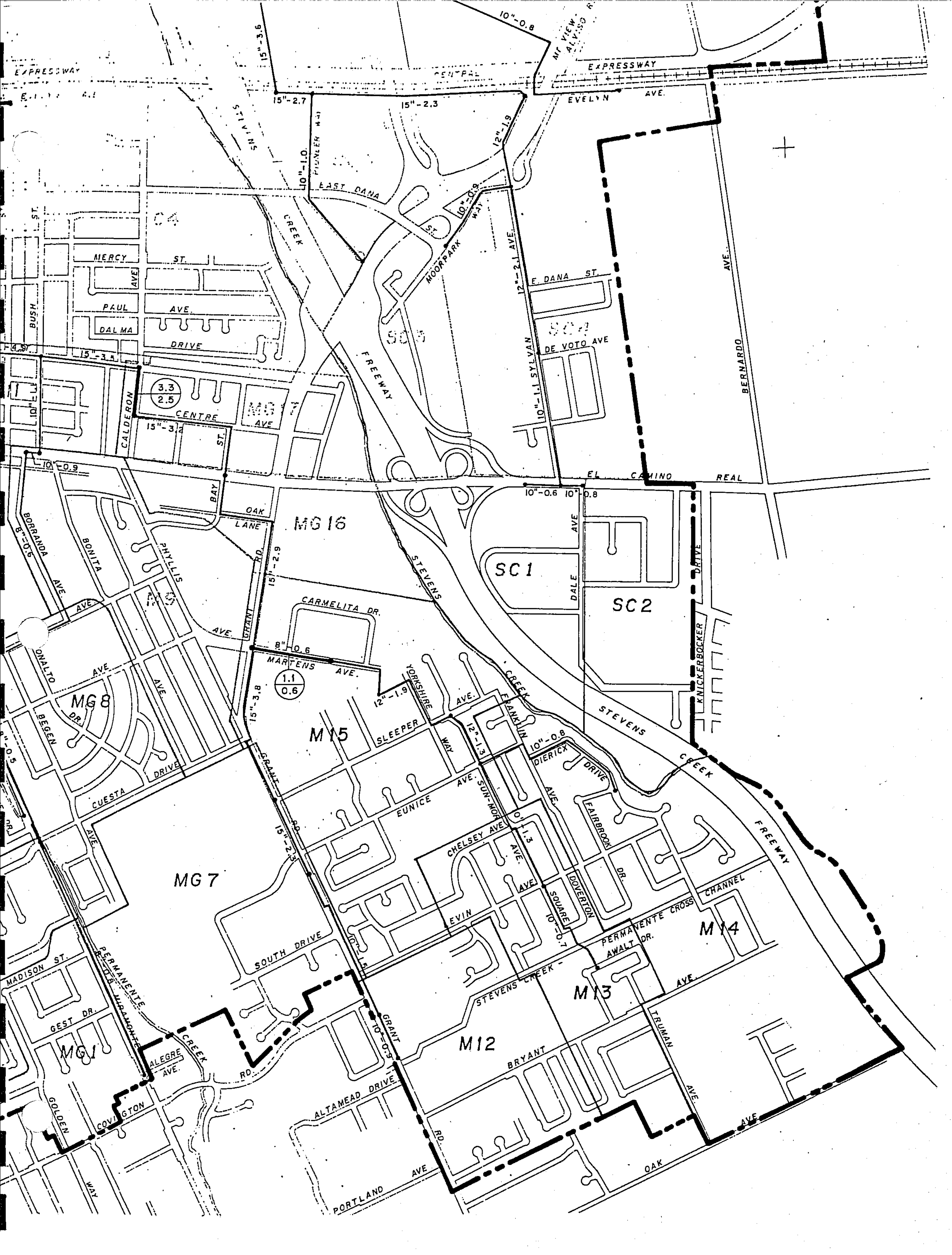


AVE. TWO DAY FLOW = .120 MGD
MAX PEAKING FACTOR = 1.36

LOGUE AVE M.H. # 18

Fig. 6-1 Sewerage Subareas and Deficiencies in Existing System, Northern Area





EXPRESSWAY
CENTRAL
EVELYN AVE.
15"-3.6
10"-0.8
15"-2.7
15"-2.3
12"-1.9
10"-1.0
EAST DANA
MOORPARK WAY
10"-0.9
C4
MERCY ST.
PAUL AVE.
DALMA DRIVE
BUSH ST.
E. DANA ST.
SC4
DE VOTO AVE
12"-2.1 AVE.
10"-1.1 SYLVAN
SC5
FREEWAY
BERNARDO AVE.
15"-3.5
3.3
2.5
CENTRE AVE
MG17
CALDERON
15"-3.2
ST.
10"-0.9
EL CAMINO REAL
10"-0.6
10"-0.8
BAY
OAK LANE
MG16
RD.
15"-2.9
BORRANDA AVE.
8"-0.6
BONITA
PHYLLIS
MG9
SC1
SC2
DALE AVE
KNICKERBOCKER DRIVE
STEVENS CREEK
CARMELITA DR.
GRANT
8"-0.6
MARTENS AVE.
1.1
0.6
12"-1.9
YORKSHIRE AVE.
FRANKLIN
ONALTO AVE.
MG8
BEGEN
DR.
15"-3.8
M15
SLEEPER
WAY
12"-1.3
10"-0.8
DIERICX DRIVE
CUESTA DRIVE
GRANT RD.
EUNICE AVE.
SUN-MOR
10"-1.3
FAIRBROOK DR.
CHELSEY AVE
AVE.
15"-2.3
MG7
SQUARE
DOVERTON
10"-0.7
LEVIN
PERMANENTE CROSS CHANNEL
M14
SOUTH DRIVE
AWALT DR.
MADISON ST.
PERMANENTE
10"-1.5
STEVENS CREEK
M13
GEST DR.
MIRAMONTE
CREEK
GRANT
10"-0.5
M12
TRUMAN AVE.
MG1
ALEGRE AVE.
RD.
BRYANT
COVINGTON
GOLDEN
ALTAMEAD DRIVE
RD.
AVE
OAK
PORTLAND AVE
WAY

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